ANALYTICAL MODEL OF COLD-FORMED STEEL FRAMED SHEAR WALL
WITH STEEL SHEET AND WOOD-BASED SHEATHING

Noritsugu Yanagi

Thesis Prepared for the Degree of

MASTER OF SCIENCE

UNIVERSITY OF NORTH TEXAS

May 2013

APPROVED:

Cheng Yu, Major Professor
Leticia Anaya, Committee Member
Reza Mirshams, Committee Member
Haifeng Zhang, Committee Member
Enrique Barbieri, Chair of the Department of Engineering Technology
Costas Tsatsoulis, Dean of the College of Engineering
Mark Wardell, Dean of the Toulouse Graduate School
Yanagi, Noritsugu. *Analytical Model of Cold-Formed Steel Framed Shear Wall with Steel Sheet and Wood-Based Sheathing*. Master of Science (Engineering Systems - Construction Management), May 2013, 171 pp., 18 tables, 24 illustrations, references, 43 titles.

The cold-formed steel framed shear walls with steel sheets and wood-based sheathing are both code approved lateral force resisting system in light-framed construction. In the United States, the current design approach for cold-formed steel shear walls is capacity-based and developed from full-scale tests. The available design provisions provide nominal shear strength for only limited wall configurations. This research focused on the development of analytical models of cold-formed steel framed shear walls with steel sheet and wood-based sheathing to predict the nominal shear strength of the walls at their ultimate capacity level.

Effective strip model was developed to predict the nominal shear strength of cold-formed steel framed steel sheet shear walls. The proposed design approach is based on a tension field action of the sheathing, shear capacity of sheathing-to-framing fastener connections, fastener spacing, wall aspect ratio, and material properties. A total of 142 full scale test data was used to verify the proposed design method and the supporting design equations. The proposed design approach shows consistent agreement with the test results and the AISI published nominal strength values.

Simplified nominal strength model was developed to predict the nominal shear strength of cold-formed steel framed wood-based panel shear walls. The nominal shear strength is determined based on the shear capacity of individual sheathing-to-framing connections, wall height, and locations of sheathing-to-framing fasteners. The proposed design approach shows a good agreement with 179 full scale shear wall test data. This analytical method requires some efforts in testing of sheathing-to-framing connections to determine their ultimate shear capacity.
However, if appropriate sheathing-to-framing connection capacities are provided, the proposed design method provides designers with an analytical tool to determine the nominal strength of the shear walls without conducting full-scale tests.
Copyright 2013

by

Noritsugu Yanagi
First and foremost, I would like to express my sincere gratitude to my adviser Dr. Cheng Yu for the financial support and mentorship throughout the two years of my graduate study, as well as his great patience and challenging but invaluable learning opportunities.

I also would like to express my appreciation to my committee members – Dr. Leticia Anaya, Dr. Reza Mirshams, and Dr. Haifeng Zhang for their support, advice, and guidance for completing my thesis.

My colleagues – Webster, Praveen, Charlie, Mark, Eslam, Marcus, and Roger, I am really thankful to you guys for the help that I received for my research and great friendship.

Finally, I am extremely grateful to my family in Japan for their support and encouragement throughout my studies in the United States.
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACKNOWLEDGEMENT</td>
<td>iii</td>
</tr>
<tr>
<td>LIST OF TABLES</td>
<td>vi</td>
</tr>
<tr>
<td>LIST OF FIGURES</td>
<td>vii</td>
</tr>
<tr>
<td>CHAPTER 1. INTRODUCTION</td>
<td>1</td>
</tr>
<tr>
<td>CHAPTER 2. PROBLEM STATEMENT AND RESEARCH OBJECTIVES</td>
<td>4</td>
</tr>
<tr>
<td>2.1 Problem Statement</td>
<td>4</td>
</tr>
<tr>
<td>2.2 Research Objectives</td>
<td>6</td>
</tr>
<tr>
<td>CHAPTER 3. ANALYTICAL MODEL AND DESIGN METHOD FOR CFS SHEAR WALLS</td>
<td>8</td>
</tr>
<tr>
<td>3.1 Literature Review</td>
<td>8</td>
</tr>
<tr>
<td>3.1.1 Previously Conducted Experimental Studies</td>
<td>8</td>
</tr>
<tr>
<td>3.1.2 Related Analytical Model - Strip Model</td>
<td>9</td>
</tr>
<tr>
<td>3.1.3 Comparison of CFS Steel Sheet Shear Walls with SPSW</td>
<td>13</td>
</tr>
<tr>
<td>3.2 Analytical Model – Effective Strip Model</td>
<td>16</td>
</tr>
<tr>
<td>3.3 Design Formula for Effective Strip Width</td>
<td>23</td>
</tr>
<tr>
<td>3.4 Discussion</td>
<td>30</td>
</tr>
<tr>
<td>3.5 Summary</td>
<td>35</td>
</tr>
<tr>
<td>CHAPTER 4. ANALYTICAL MODEL AND DESIGN METHOD FOR CFS SHEAR WALLS</td>
<td>36</td>
</tr>
<tr>
<td>4.1 Literature Review</td>
<td>36</td>
</tr>
<tr>
<td>4.1.1 Previously Conducted Experimental Studies</td>
<td>36</td>
</tr>
</tbody>
</table>
4.1.2 Related Analytical Models ................................................................. 40
4.1.3 Capacity of Individual Sheathing-to-Framing Connection .......... 43
4.1.4 Chen (2004) – Simplified Strength Model (SSM) ......................... 45
4.1.5 Chen (2004) – Comparison of Shear Wall Capacity ....................... 52
4.2 Screw Connection Test – UNT Shear Wall Testing Program .......... 54
  4.2.1 Test Setup ..................................................................................... 55
  4.2.2 Test Specimen .............................................................................. 56
  4.2.3 Test results ................................................................................... 58
4.3 Effective Strip Model ......................................................................... 59
4.4 Simplified Nominal Strength Model .................................................. 64
4.5 Discussion ......................................................................................... 67
4.6 Summary ......................................................................................... 70

CHAPTER 5. CONCLUSIONS AND RECOMMENDATIONS .................. 71
  5.1 Steel Sheet Shear Wall ................................................................. 71
  5.2 Wood-based Panel Shear Wall ...................................................... 72
  5.3 Recommendations for Future Research ......................................... 73

APPENDIX A DESIGN EXAMPLE OF CFS-STEEL SHEET SHEAR WALL .......... 75
APPENDIX B DESIGN EXAMPLE OF CFS-WOOD-BASED PANEL SHEAR WALL ........ 80
APPENDIX C DATA SHEETS OF OSB AND PLYWOOD CONNECTION TESTS ........ 83
APPENDIX D DATA SHEETS OF PLYWOOD SHEATHED SHEAR WALL TESTS ........ 156
REFERENCES ......................................................................................... 167
## LIST OF TABLES

<table>
<thead>
<tr>
<th>Table Number</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Table 2.1</td>
<td>Nominal strength of CFS shear walls for wind design (lbs/ft) (AISI, 2007)</td>
<td>4</td>
</tr>
<tr>
<td>Table 2.2</td>
<td>Nominal strength of CFS shear walls for seismic design (lbs/ft) (AISI, 2007)</td>
<td>5</td>
</tr>
<tr>
<td>Table 3.1</td>
<td>Comparison of nominal shear strength values (strip model)</td>
<td>14</td>
</tr>
<tr>
<td>Table 3.2</td>
<td>List of test data label (steel sheet shear wall)</td>
<td>24</td>
</tr>
<tr>
<td>Table 3.3</td>
<td>Measured material properties (steel sheet shear wall)</td>
<td>26</td>
</tr>
<tr>
<td>Table 3.4</td>
<td>Statistical results of nominal shear strength values</td>
<td>30</td>
</tr>
<tr>
<td>Table 3.5</td>
<td>Comparison of nominal shear strength values (effective strip model)</td>
<td>32</td>
</tr>
<tr>
<td>Table 3.6</td>
<td>Summary of resistance factors and safety factors (effective strip model)</td>
<td>34</td>
</tr>
<tr>
<td>Table 4.1</td>
<td>Monotonic test results of sheathing-to-framing connection (Okasha, 2004)</td>
<td>44</td>
</tr>
<tr>
<td>Table 4.2</td>
<td>Cyclic test result of sheathing to framing connection (Okasha, 2004)</td>
<td>44</td>
</tr>
<tr>
<td>Table 4.3</td>
<td>Comparison of testing-to-predicted shear capacity ratio (Chen, 2004)</td>
<td>53</td>
</tr>
<tr>
<td>Table 4.4</td>
<td>Test matrix of sheathing-to-framing connection test</td>
<td>57</td>
</tr>
<tr>
<td>Table 4.5</td>
<td>Result of sheathing-to-framing connection tests</td>
<td>58</td>
</tr>
<tr>
<td>Table 4.6</td>
<td>List of test data label (wood-based panel shear wall)</td>
<td>61</td>
</tr>
<tr>
<td>Table 4.7</td>
<td>Measured material properties (wood-based panel shear wall)</td>
<td>62</td>
</tr>
<tr>
<td>Table 4.8</td>
<td>Statistical results of nominal shear strength values</td>
<td>67</td>
</tr>
<tr>
<td>Table 4.9</td>
<td>Comparison of nominal shear strength values</td>
<td>68</td>
</tr>
<tr>
<td>Table 4.10</td>
<td>Summary of resistance factor and safety factor (simplified nominal strength model)</td>
<td>70</td>
</tr>
</tbody>
</table>
# LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure 1.1</td>
<td>CFS shear walls using steel sheet sheathing (courtesy of Simpson Strong-Tie®)</td>
<td>2</td>
</tr>
<tr>
<td>Figure 1.2</td>
<td>Components in a typical CFS shear wall</td>
<td>3</td>
</tr>
<tr>
<td>Figure 3.1</td>
<td>Steel structure with SPSW system (courtesy of Magnusson Klemencic Associates)</td>
<td>10</td>
</tr>
<tr>
<td>Figure 3.2</td>
<td>Components in typical SPSW</td>
<td>10</td>
</tr>
<tr>
<td>Figure 3.3</td>
<td>Diagonal tension field action of SPSW (Berman and Bruneau, 2005)</td>
<td>11</td>
</tr>
<tr>
<td>Figure 3.4</td>
<td>Strip model of SPSW</td>
<td>11</td>
</tr>
<tr>
<td>Figure 3.5</td>
<td>Tension field action of CFS steel sheet shear walls</td>
<td>13</td>
</tr>
<tr>
<td>Figure 3.6</td>
<td>Connection failure of CFS steel sheet shear wall</td>
<td>14</td>
</tr>
<tr>
<td>Figure 3.7</td>
<td>Definition of shear wall configuration</td>
<td>15</td>
</tr>
<tr>
<td>Figure 3.8</td>
<td>Effective strip model of steel sheet sheathing</td>
<td>17</td>
</tr>
<tr>
<td>Figure 3.9</td>
<td>Equilibrium of nominal tension force in sheathing and sum of nominal connection shear capacity</td>
<td>19</td>
</tr>
<tr>
<td>Figure 3.10</td>
<td>Sheathing-to-framing fastener connection layout within effective strip</td>
<td>22</td>
</tr>
<tr>
<td>Figure 3.11</td>
<td>Definition of test data label (steel sheet shear wall)</td>
<td>25</td>
</tr>
<tr>
<td>Figure 3.12</td>
<td>Maximum width of effective strip</td>
<td>27</td>
</tr>
<tr>
<td>Figure 3.13</td>
<td>Comparison of proposed design curve with test results (actual material properties)</td>
<td>28</td>
</tr>
<tr>
<td>Figure 3.14</td>
<td>Comparison of proposed design curve with test results (nominal material properties)</td>
<td>29</td>
</tr>
<tr>
<td>Figure 4.1</td>
<td>Force distribution in frame members (Chen, 2004)</td>
<td>47</td>
</tr>
<tr>
<td>Figure 4.2</td>
<td>Force distribution in sheathing-to-framing connections (Chen, 2004)</td>
<td>47</td>
</tr>
<tr>
<td>Figure 4.3</td>
<td>Typical set up of a test specimen</td>
<td>56</td>
</tr>
</tbody>
</table>
Figure 4.4 Close-up of a connection test specimen .................................................................57
Figure 4.5 Definition of test ID (connection test).................................................................58
Figure 4.6 Definition of test data label (wood-based panel shear wall) ...............................62
Figure 4.7 Effective ratio versus screw spacing ..................................................................63
Figure 4.8 Comparison of proposed design curve with test results .................................66
CHAPTER 1

INTRODUCTION

The use of cold-formed steel (CFS) framing in low and mid-rise building construction has become very popular in recent years due to its many beneficial aspects. Some of its desirable traits include cost-effectiveness, non-combustibility, recyclability, and excellence in material consistency. Also, CFS has high durability and high strength despite the fact that it is significantly lighter than traditional framing materials such as concrete and hot-rolled steel. The use of CFS as structural framing members is a great choice for up to medium rise commercial and public construction such as schools, office buildings, apartments, and hotels.

In building construction, CFS structural members are classified into two major categories: individual structural framing members and panels and decks. The primary roles of individual structural members are to transfer loads to the ground and provide structural strength and stiffness to the building. Panels and decks also resist in-plane and out-of-plane loads, but they also act as essential surface component of the building such as floors, walls, and roofs (Yu and LaBoube, 2010).

Along with CFS framing, steel sheets and wood-based panels are used as sheathing materials to provide lateral strength and stiffness to the framing. This type of structural component is called shear wall and is widely employed in CFS construction as a lateral force resisting system against wind and earthquake forces. Figure 1.1 shows a three-story residential building using CFS steel sheet shear wall.
Typical CFS shear walls consist of framing members (studs and tracks), sheathing, fasteners, and hold-downs. Studs and tracks are vertical and horizontal framing members respectively. Sheathing is fastened to the boundary framing members and the inner stud by self-drilling self-tapping screws. Plywood, oriented strand board (OSB), and steel sheet are the most popular sheathing materials. Hold-downs are installed on the boundary studs with screws, and then the hold-downs are anchored to the foundation or footing of the structure by anchor bolts. Figure 1.2 shows a typical 8 ft. × 4 ft. CFS steel sheet shear wall and its components. Overturning and shear forces are the two forces acting on CFS shear walls when a lateral force is applied at the top of the wall. Overturning force is resisted by the hold-downs and the anchor bolts. In-plane shear force is resisted by the sheathing and sheathing-to-framing screw connections.
Figure 1.2 Components in a typical CFS shear wall
CHAPTER 2
PROBLEM STATEMENT AND RESEARCH OBJECTIVES

2.1 Problem Statement

International Building Code (IBC 2006) and North American Standard for Cold-Formed Steel Framing – Lateral Design (AISI S213, 2007) provide provisions for the design of CFS shear walls. In these provisions, tabulated nominal shear strength values are presented for three types of sheathing materials: 15/32 in. structural I plywood, 7/16 in. OSB, and 0.018 in. and 0.027 in. steel sheet. Those published values are based on full-scale shear wall tests conducted by Serrette et al. (1996, 1997, and 2002). Table 2.1 and Table 2.2 show the tables of nominal strength from AISI S213 (2007) for wind and seismic loads respectively. The tables are also adopted by IBC (2006).

Table 2.1 Nominal strength of CFS shear walls for wind design (lbs/ft) (AISI, 2007)

<table>
<thead>
<tr>
<th>Assembly Description</th>
<th>Maximum Aspect Ratio (h/w)</th>
<th>Fastener Spacing at Panel Edges (inches)</th>
<th>Designation Thickness of Stud, Track and Blocking (mils)</th>
<th>Required Sheathing Screw Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>15/32” Structural I sheathing (4-ply), one side</td>
<td>2:1</td>
<td>1065</td>
<td>-</td>
<td>33</td>
</tr>
<tr>
<td>7/16” rated sheathing (OSB), one side</td>
<td>2:1</td>
<td>910</td>
<td>1410</td>
<td>1735</td>
</tr>
<tr>
<td>7/16” rated sheathing (OSB), one side oriented perpendicular to framing</td>
<td>2:1</td>
<td>1020</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>7/16” rated sheathing (OSB), one side</td>
<td>4:1</td>
<td>-</td>
<td>1025</td>
<td>1425</td>
</tr>
<tr>
<td>0.018” steel sheet, one side</td>
<td>2:1</td>
<td>485</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>0.027” steel sheet, one side</td>
<td>4:1</td>
<td>-</td>
<td>1000</td>
<td>1085</td>
</tr>
<tr>
<td></td>
<td>2:1</td>
<td>647</td>
<td>710</td>
<td>778</td>
</tr>
</tbody>
</table>
Table 2.2 Nominal strength of CFS shear walls for seismic design (lbs/ft) (AISI, 2007)

<table>
<thead>
<tr>
<th>Assembly Description</th>
<th>Maximum Aspect Ratio (h/w)</th>
<th>Fastener Spacing at Panel Edges (inches)</th>
<th>Designation Thickness of Stud, Track and Blocking (mils)</th>
<th>Required Sheathing Screw Size</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>6</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>15/32” Structural 1 sheathing (4-ply), one side</td>
<td>2:1</td>
<td>780</td>
<td>990</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>2:1</td>
<td>890</td>
<td>1330</td>
<td>1775</td>
</tr>
<tr>
<td></td>
<td></td>
<td>68</td>
<td>10</td>
<td>68</td>
</tr>
<tr>
<td>7/16” OSB, one side</td>
<td>4:1</td>
<td>700</td>
<td>915</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>4:1</td>
<td>825</td>
<td>1235</td>
<td>1545</td>
</tr>
<tr>
<td></td>
<td>2:1</td>
<td>940</td>
<td>1410</td>
<td>1760</td>
</tr>
<tr>
<td></td>
<td>2:1</td>
<td>1232</td>
<td>1848</td>
<td>2310</td>
</tr>
<tr>
<td>0.018” steel sheet, one side</td>
<td>2:1</td>
<td>390</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>0.027” steel sheet, one side</td>
<td>4:1</td>
<td>-</td>
<td>1000</td>
<td>1085</td>
</tr>
<tr>
<td></td>
<td>2:1</td>
<td>647</td>
<td>710</td>
<td>778</td>
</tr>
</tbody>
</table>

The current CFS design provisions are capacity based design and provide no analytical methodology to predict the shear resistances of CFS shear walls. No analytical models or design equations have been incorporated into design provisions for predicting the shear strength of CFS shear walls. Instead, those provisions only provide nominal shear strength values for specified and limited wall configurations. When designing CFS shear walls, it is apparent that with the current design specifications, structural engineers have limited options in sheathing materials, sheathing thickness, wall aspect ratios, etc. Oftentimes engineers are forced to conduct full-scale shear wall testing to determine nominal shear strength of shear wall configurations that are not listed in the provisions before incorporating the design to use. Contrarily, analytical models and closed-form design equations for the hot-rolled steel plate shear wall (SPSW) and reinforced concrete shear wall have been developed and adopted by design provisions (AISC Seismic Design Manual, 2005; ACI Building Code Requirements 318, 2005). In order to provide more design options to the engineers without conducting full-scale testing, it is necessary that analytical models and design equations be developed to predict nominal shear strength of CFS shear walls with steel sheet and wood-based panel sheathing.
2.2 Research Objectives

The purpose of this research is to develop an analytical method to predict nominal shear strength of CFS shear walls with steel sheet and wood-based panel sheathing. Experimental data from previously conducted full-scale shear wall testing programs is incorporated into this research to analyze dominant failure mechanism of CFS shear wall systems. An analytical model will be developed based on the identified failure mechanism and an engineering mechanics approach. Since steel sheets and wood-based panels have completely different failure mechanism and material properties, the development of the analytical models is conducted separately. Thus, this research is divided into two parts. An analytical model of steel sheet shear wall will be developed first. Once the model is developed, the analytical method for the design of steel sheet shear walls will be applied to wood-based panel shear walls to see if the method is also appropriate for wood-based panel shear walls. Otherwise a different analytical model will be developed. The specific research objectives of this thesis are listed below:

For steel sheet shear walls,

i). Incorporate test data from Yu et al. (2007, 2009) and Balh (2010), obtain necessary information to determine the dominant failure mechanism of shear wall system and contributing factor to achieve its peak resistance;

ii). Develop an analytical model and design equations for steel sheet shear wall;

iii). Seek for an agreement between shear wall peak resistance from the test data and estimated nominal strength values from the analytical approach;

iv). Recommend an efficient analytical model for steel sheet shear wall and perform reliability analysis to determine the Load and Resistance Factor Design (LRFD)
resistance factor and Allowable Stress Design (ASD) safety factor for both wind and seismic design.

For wood-based panel shear walls,

v). Perform OSB-to-framing and plywood-to-framing connection test to determine the capacity of individual sheathing-to-framing connection;

vi). Incorporate test data from Branston (2004), Chen (2004), Boudreault (2005), Blais (2006), Rokas (2006), and Li (2012);

vii). Apply the analytical method developed for steel sheet shear walls to see if the analytical model is also applicable to wood-based panel shear walls;

viii). Review an analytical model proposed by Chen (2004) and apply the model to larger pool of test data to seek for an agreement between shear wall peak resistance from the test data and estimated nominal strength values from the analytical model;

ix). Recommend an efficient analytical model for wood-based panel shear wall and perform reliability analysis to determine the LRFD resistance factor and ASD safety factor for both wind and seismic design.
CHAPTER 3

ANALYTICAL MODEL AND DESIGN METHOD FOR CFS SHEAR WALLS USING STEEL SHEET SHEATHING

3.1 Literature Review

In this section, brief overview of previously conducted experimental studies on steel sheet shear walls is presented. Also, an existing analytical model of hot-rolled steel plate shear is reviewed to seek its applicability to CFS steel sheet shear walls.

3.1.1 Previously Conducted Experimental Studies

As mentioned earlier, through literature review, no analytical methodology has been developed to determine the nominal shear strength of CFS steel sheet shear walls. However, some researchers have conducted experimental investigations to capture the characteristics and behavior of CFS steel sheet shear walls (Serrette, 1997; Yu, 2007, 2009; Balh, 2010). A full-scale testing program was sponsored by the American Iron and Steel Institute (AISI) and conducted at Santa Clara University by Serrette (1997) in order to determine nominal shear strength values for steel sheet shear walls. The total of 12 steel sheet sheathed shear walls were tested, in which six of them were monotonic and the other six were cyclic tests. In this test program, wall configuration was limited: height to width wall aspect ratio of 2.0 (8 ft. × 4 ft.) and 4.0 (8 ft. × 2 ft.), framing thickness of 33 mil, and sheathing thickness of 18 mil and 27 mil. It was noted that all the shear wall specimens experienced some form of failure at sheathing to framing connections especially at the corners of the walls. Yu et al. (2007, 2009) conducted two-phase full-scale testing of steel sheet shear walls at University of North Texas to expand the available nominal strength values for more wall configurations. In phase 1, the main purpose was to provide nominal strength values for the walls with 30 mil and 33 mil steel sheet sheathing. 27 mil sheathing was also used for some specimens to replicate the results of test program by...
Serrette (1997). Each wall type had 2 in., 4 in., and 6 in. edge fastener spacing, and in total, 33 monotonic and 33 cyclic tests were carried out. Yu et al. (2007) reported that for the wall specimens with 4 in. and 6 in. edge fastener spacing, dominant mode of failure was a combination of buckling of steel sheet sheathing and fastener pull-out from the framing. For the walls with 2 in. fastener spacing, dominant mode of failure was a combination of buckling of steel sheet sheathing and flange distortional buckling of boundary studs. Yu et al. (2009) conducted additional shear wall testing with 18 mil and 27 mil steel sheet shear walls to verify the published shear strength values in AISI S213 (2007). Also, Yu et al. (2009) conducted testing of steel sheet shear walls with aspect ratio of 1.33 (8’x6’) to investigate appropriate seismic detailing in framing and joint to the sheathing. Balh (2010) was involved in full-scale testing of steel sheet shear walls at McGill University in Canada, and the result of comprehensive analysis of the test data was reported in Balh (2010). Balh (2010) also reported sheathing-to-framing connection failure at the corners of the walls and steel sheet buckling were the dominant failure modes.

3.1.2 Related Analytical Model - Strip Model

Hot-rolled steel plate shear wall (SPSW) has been used as lateral load resisting systems for buildings in North America and Japan for the last 30 years. During this time, SPSW has been studied experimentally and analytically by a number of researchers (Thorburn et al., 1983; Timler and Kulak, 1983; Tromposch and Kulak, 1987; Roberts and Sabouri-Ghomi, 1992; Sabouri-Ghomi and Roberts, 1992; Cassese et al., 1993; Elgaaly et al., 1993; Driver et al., 1998; Elgaaly and Liu, 1997; Elgaaly 1998; Rezai, 1999; Lubell et al., 2000; Berman and Bruneau, 2004, Vian and Bruneau, 2004). Figure 3.1 shows a hot-rolled steel structure using SPSWs as lateral force resisting systems.
A typical SPSW system (Figure 3.2) consists of horizontal vertical boundary elements (columns), horizontal boundary elements (beams), and infill plates (thin steel plates). The infill plates are welded all around to the boundary members.

When lateral forces are applied to SPSW structure, infill plates are subjected to out-of-plane shear buckling, and multiple diagonal tension fields appear within the infill plates (Figure 3.3). The energy from the loading is designed to dissipate within the infill plates through the tension field action of the infill plates. On the other hand, the boundary members are designed to act elastic throughout the loading.
Based on the elastic strain energy assumption, Thorburn et al. (1983) developed an analytical model known as a strip model (Figure 3.4). In strip model, the infill plate is divided into a series of inclined strip elements. The inclination of the strip members, $\alpha$ is oriented in the same direction as the direction of the tensile forces experienced in the panel, and the strips are capable of transferring only tension forces. Individual strip members have identical cross section area as the product of the width and thickness of the strip.

With respect to development of the analytical model, Thorburn et al. (1983) derived an equation to determine the angle of inclination of the tension strips based on the principle of least work. In the process of the derivation of the formula of angle of inclination by Thorburn et al. (1983), both boundary members were considered rigid. However, Timler and Kulak (1983) re-
evaluated this formula to include the effect of bending of the vertical boundary elements. The influence of the horizontal boundary members was excluded from the derivation of the formula since the tension field created above and below the horizontal boundary members oppose to each other and cancel out the influence to the infill plates. Based on the strip model, Berman and Bruneau (2003b) derived an equation to determine the ultimate strength of SPSW using plastic analysis.

The angle of inclination of the tension strips is given by:

\[
\alpha = \tan^{-1} \left( \frac{1+\frac{t L}{2 A_c}}{1+\frac{1}{h A_c} + \frac{h^3}{360 I_c L}} \right)
\]  

(3.1)

where:

\( t \) = thickness of the infill plate;

\( h \) = height of the wall;

\( L \) = width of the wall;

\( I_c \) = moment of inertia of the vertical boundary element;

\( A_c \) = cross section area of the vertical boundary element;

\( A_b \) = cross section area of the horizontal boundary element.

The ultimate strength of single story SPSW is given by:

\[ V = \frac{1}{2} F_y t L \sin 2\alpha \]  

(3.2)

where:

\( V \) = horizontal shear force applied to the wall;

\( F_y \) = yield stress of the infill plate.

The strip model and the design equations were adopted by BSSC (2004) and AISC (2005).
3.1.3 Comparison of CFS Steel Sheet Shear Walls with SPSW

The behavior of CFS steel sheet shear walls similar behaviors as SPSWs to some extent. Both structures demonstrated out-of-plane shear buckling and create diagonal tension field in the sheathing or infill plate. Diagonal tension field action of SPSW is previously presented in Figure 3.3. Figure 3.5 shows diagonal tension field action of CFS steel sheet shear walls with different aspect ratio observed in Yu (2007, 2009).

![Figure 3.5 Tension field action of CFS steel sheet shear walls](image)

According to strip model, the ultimate strength of SPSW is determined when an infill plate reaches its yielding capacity. However, the boundary conditions of SPSW and CFS shear walls are different, and thus different types of failure modes are observed in CFS steel sheet shear wall. The infill plate in SPSW is usually welded to the boundary elements while CFS steel sheet sheathing is generally fastened to the boundary elements by self-drilling screws. In addition to the sheathing shear buckling, commonly observed failure modes of CFS steel sheet shear wall include sheathing-to-framing fastener pull-out, pull-over, and sheathing tear at the locations of the fasteners. Figure 3.6 shows different types of connection failure of CFS steel sheet shear
walls. These failures do not exist in SPSW and must be considered when predicting the shear strength of CFS shear walls.

Figure 3.6 Connection failure of CFS steel sheet shear wall

In order to verify the applicability of strip model design approach to CFS steel sheet shear wall, published nominal shear strength of CFS sheet steel shear walls from Table C2.1-1 (wind) and Table C2.1-3 (seismic) in AISI S213 (2007) are used to compare with the nominal shear strength values calculated based on the strip model and equation 3.2. A total of eight shear wall configurations are analyzed. Table 3.1 shows the comparison of published nominal shear strength values with the ones estimated by using strip model approach.

Table 3.1 Comparison of nominal shear strength values (strip model)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>2:1x33x18-6</td>
<td>485</td>
<td>390</td>
<td>1728</td>
</tr>
<tr>
<td>4:1x43x27-4</td>
<td>1000</td>
<td>1000</td>
<td>1935</td>
</tr>
<tr>
<td>4:1x43x27-3</td>
<td>1085</td>
<td>1085</td>
<td>1935</td>
</tr>
<tr>
<td>4:1x43x27-2</td>
<td>1170</td>
<td>1170</td>
<td>1935</td>
</tr>
<tr>
<td>2:1x33x27-6</td>
<td>647</td>
<td>647</td>
<td>2531</td>
</tr>
<tr>
<td>2:1x33x27-4</td>
<td>710</td>
<td>710</td>
<td>2531</td>
</tr>
<tr>
<td>2:1x33x27-3</td>
<td>778</td>
<td>778</td>
<td>2531</td>
</tr>
<tr>
<td>2:1x33x27-2</td>
<td>845</td>
<td>845</td>
<td>2531</td>
</tr>
</tbody>
</table>
In Table 3.1, the first column from the left lists wall configurations included in AISI S213 (2007), the second and third columns list the published nominal shear strength of CFS steel sheet shear walls for wind and seismic loads respectively, and the fourth column lists the nominal shear strength values estimated by using strip model design approach. The definition of wall configuration is illustrated in Figure 3.7.

![Figure 3.7 Definition of shear wall configuration](image)

The grade of steel sheet sheathing and framing members is considered to be ASTM A1003 Grade 33, having minimum yield strength of 33 ksi and tensile strength of 45 ksi. Design values are used for sheathing thicknesses and cross-sectional area and moment of inertia of boundary members. According to the results shown in Table 3.1, all of the estimated nominal shear strength values are significantly higher than the published values. This is a good indication that the yielding of steel sheathing is not likely the primary mode of failure for CFS steel sheet shear wall. Also, according to strip model approach, screw spacing does not affect the nominal strength values as long as all the other design parameters remain the same. However, this is not true in reality, and the published values indicate that the nominal strength is higher as the screw spacing gets closer. Strip model assumes that material of infill plate or sheathing is the only failure of the shear wall system, and it also fails to capture the important trend that are obvious in CFS steel sheet shear walls. Thus, the strip model cannot be applied to CFS steel sheet shear walls to estimate its nominal shear strength. A new analytical model has to be developed in order
to include sheathing-to-framing connection failure as one of the failure modes and capture the effect of screw spacing.

3.2 Analytical Model – Effective Strip Model

As mentioned in the previous section, steel sheet sheathing of CFS steel sheet shear walls (SSSWs) exhibit tension field action under lateral loading, and the shear resistance of CFS-SSSWs was primarily provided by the steel sheathing through the diagonal tension field action. The dominant failure mechanism was screw connection failure within the diagonal tension field. As illustrated in Figure 3.5, the steel sheathing is not equally contributing to the shear resistance across the width of the entire shear wall. It was observed that a partial width of the sheathing was accountable for conveying most of the tension force in the system. Also, in most tested wall specimens, sheathing-to-framing connection failure occurred at the corners of the shear walls normally inside the observed tension field. This observation led to the creation of an analytical model - effective strip model, for predicting the shear strength of CFS-SSSWs as illustrated in Figure 3.8. In the effective strip model, it is assumed that a particular width of the sheathing in the diagonal direction – the effective strip is involved in the tension field action to provide shear resistance to the lateral force which is applied to the top of the wall.
Figure 3.8 Effective strip model of steel sheet sheathing

In Figure 3.8, \( V_a \) is the applied lateral load, \( T \) is the resulting tension force in the effective strip of the sheathing, and \( h \) and \( W \) are the height and the width of the wall respectively. \( \alpha \) is the angle at which the tension force is acting. \( W_e \) is the width of the effective strip that is accountable for conveying all the tension force in the system and is defined in a way that it is perpendicular to the direction of the strip. It is assumed that the effective strip is centered to the diagonal line from the corner to the other corner of the wall. Based on the effective strip model, the applied lateral load \( V_a \) can be expressed in the following equation.

\[
V_a = T \cos \alpha
\]  

(3.3)
In this model, the applied lateral load is directly related to the tension force experienced in the effective strip of the steel sheet sheathing. In other words, the maximum force obtained from shear wall system is limited by the maximum tension force in the sheathing. The maximum tension force in the sheathing is then limited by capacities of two components in the system. The first component is the capacity of sheathing-to-framing connection at both ends of the effective strip (e.g. the corners of shear walls inside the effective tension field). The second component is the material yield strength of the effective strip. The yielding of the sheathing material was not observed in the actual experimental investigation by Yu (2007, 2009). However, this type of failure mode could possibly happen when a large number of fasteners are used to connect the sheathing to the CFS frame. Thus, the nominal shear force in a CFS-SSSW can be determined as follows.

\[ V_n = T_n \cos \alpha \] (3.4)

where \( V_n \) is the nominal shear strength of a CFS-SSSW and \( T_n \) is the nominal tension strength of the effective strip of the sheathing. As previously discussed, the nominal tension force is determined as follows.

\[ T_n = \text{minimum}\left\{ \sum_{i=1}^{n} P_{nsi}, W_e t_{sh} F_y \right\} \] (3.5)

where \( P_{ns} \) is the nominal shear strength of individual sheathing-to-framing connection, \( t_{sh} \) is the sheathing thickness, \( F_y \) is the sheathing yield stress, and \( n \) is the total number of fasteners at one end of the effective strip. The proposed model assumes that the fastener configurations are equivalent at both ends of the effective strip. When the fastener configurations are different at the two ends of the effective strip, the fastener strength on the weaker end shall be used in Equation 3.5. The nominal tension force \( T_n \) is determined as the smaller of the sum of the nominal shear
strengths of sheathing-to-framing connections and the material yield strength of the effective strip of sheathing. Figure 3.9 illustrates the equilibrium of the tension force in sheathing and the sum of connection shear strength.

![Equilibrium of nominal tension force in sheathing and sum of nominal connection shear capacity](image)

Figure 3.9 Equilibrium of nominal tension force in sheathing and sum of nominal connection shear capacity

Nominal shear strength of fastener connections is limited by three types of failure mechanisms. The first is connection shear limited by tilting and bearing. The second is connection shear limited by end distance measured in line of force from center of a standard hole to the nearest end of connected parts. The third is shear failure in screws. The consideration of those three types of fastener failures is consistent with the fastener provisions in the North American Specification for the Design of Cold-Formed Steel Structural Members (AISI S100, 2007). Thus, sheathing-to-framing connection shear strengths are determined based on the smallest value obtained from calculation by following E4.3.1, E4.3.2, and E4.3.3 of AISI S100 (2007).
Connection capacity based on tilting and bearing can be determined based on section E.4.3.1 of AISI S100 (2007) and for \( t_2/t_1 \leq 1.0, \)

\[
P_{ns} = \text{minimum}\left\{ 4.2(t_2^3d)^{1/2}F_{u_2}, 2.7t_1dF_{u_1}, 2.7t_2dF_{u_2} \right\}
\]  

(3.6)

where \( t_1 \) is the thickness of member in contact with screw head or washer, \( t_2 \) is the thickness of member not in contact with screw head or washer, \( d \) is the nominal screw diameter, \( F_{u_1} \) is the tensile strength of member in contact with screw head or washer, and \( F_{u_2} \) is the tensile strength of member not in contact with screw head or washer. For \( t_2/t_1 \geq 2.5, \)

\[
P_{ns} = \text{minimum}\{2.7t_1dF_{u_1}, 2.7t_2dF_{u_2} \}
\]

(3.7)

For \( 1.0 < t_2/t_1 < 2.5, \) \( P_{ns} \) shall be calculated by linear interpolation between the above two cases.

Connection capacity limited by end distance can be determined based on section E.4.3.2.

\[
P_{ns} = teF_u
\]

(3.8)

where \( t \) is the thickness of part in which end distance is measured, \( e \) is the distance measured in line of force from center of a standard hole to nearest end of connected part, and \( F_u \) is the tensile strength of part in which end distance is measured. The end distance \( e \) can be determined based on one half of the flange width of stud or track and aspect ratio of a shear wall.

Connection strength limited by shear in screw is generally provided by the manufacturer or determined by tests. The provision of E4.3.3 in AISI S100 (2007) does not provide design equations for shear strength in screw. It is worth noting that the screw’s pull-out and pull-over strength do affect the shear wall’s shear strength. However those forces are out-of-plane, not directly contributing to the in-plane shear strength, therefore the limit states of screw’s pull-out and pull-over are not considered in the calculation of \( P_{ns}. \)
Equations 3.4 and 3.5 express the key concept of the Effective Strip Model for calculating the shear strength of CFS-SSSWs. The nominal shear strength of a CFS-SSSW can be further represented in terms of the number of sheathing-to-framing connections and the connection strength within its effective strip. By substituting Eq. 3.5 into Eq. 3.4 and considering practical construction details (shown in Figure 3.9), the following equation can be obtained.

\[
V_n = \text{minimum} \left\{ (n_t P_{ns,t} + n_s P_{ns,s} + P_{ns,t&s}) \cos \alpha, W_e t F_y \cos \alpha \right\}
\]  

(3.9)

where \( n_t \) is the number of fasteners on tracks within the effective strip at one end, \( n_s \) is the number of fasteners on boundary studs within the effective strip at one end, \( P_{ns} \) is the nominal shear strength of the fasteners, the subscript \( t \) and \( s \) are regarding connections on track and stud respectively, and the subscript \( t&s \) is regarding a fastener at the corner of the wall at which its fastener is penetrating through sheathing, track, and stud.

Equation 3.9 summarizes the proposed effective strip model for predicting the nominal shear strength of a CFS-SSSW. Based on the geometry shown in Figure 3.10, the number of connections can be related to effective strip width.
Figure 3.10 Sheathing-to-framing fastener connection layout within effective strip

In Figure 3.10, $s$ is the fastener spacing (assuming that the fastener spacing is uniform on the panel edges) and $l_t$ is the approximate length on track that is contributing to the effective tension strip determined by the product of the number of fasteners on track within its effective width and the fastener spacing. Likewise, $l_s$ is the approximate contributing length on stud and determined by the product of the number of fasteners on stud within its effective width and the fastener spacing. The effective strip width of sheathing can be expressed as follows.

$$W_e = 2l_t \sin \alpha = 2sn_t \sin \alpha \text{ or } W_e = 2l_s \cos \alpha = 2sn_s \cos \alpha$$  \hspace{1cm} (3.10)

In these equations, the short distances of the fastener at the corner to the outer face of stud and to the outer face of track are not included in $l_t$ and $l_s$ respectively. Inclusion of these short distances will complicate the equations, and also, the deviations due to the exclusion of these short distances are considered to be minimal. The number of the fasteners on track within its effective width can be described as following equations.

$$n_t = \frac{w_e}{2s \sin \alpha}$$  \hspace{1cm} (3.11)
Likewise, the number of fasteners on stud can be expressed in the form of the following as well.

\[ n_s = \frac{W_e}{2s \cos \alpha} \]  

(3.12)

Note that the number of fasteners on stud to the number of fasteners on track ratio gives the tangent of an angle \( \alpha \), which is the height to width aspect ratio of the shear wall. Substituting the number of fasteners on track and stud within its effective width to the previously defined equation of nominal shear strength of a CFS steel sheet shear walls, the equation becomes as follows.

\[ V_n = \text{minimum} \left\{ \left( \frac{W_e}{2s \sin \alpha} P_{ns,t} + \frac{W_e}{2s \cos \alpha} P_{ns,s} + P_{ns,t\&s} \right) \cos \alpha, W_e t_s h F_y \cos \alpha \right\} \]  

(3.13)

The proposed Effective Strip Model requires the knowledge of the width of the effective strip in order to obtain the shear strength of CFS-SSSWs. The following section emphasizes the work of developing closed-form formula of \( W_e \).

3.3 Design Formula for Effective Strip Width

Based on the proposed effective strip model, nominal shear strength of a CFS steel sheet shear walls can be calculated in terms of nominal shear capacity of sheathing-to-framing connections and material yield capacity of an effective strip once the effective width of the tension strip is determined. In order to do so, design formula had to be developed to predict the effective strip width of different configurations of CFS steel sheet shear walls. Experimental testing data of 142 monotonic and cyclic tests of CFS steel sheet shear walls from Yu et al. (2007, 2009) and Balh (2010) are analyzed to develop and verify the formula of the effective strip width. The 142 tests, 70 monotonic and 72 cyclic, cover a large range of variations in the wall configurations which include framing thickness 33 mil to 68 mil, steel sheathing thickness 18 mil
to 33 mil, fastener spacing 2 inches to 6 inches, and wall aspect ratio 1.0 to 4.0. The list of test
data used in this research is presented in Table 3.2.

<table>
<thead>
<tr>
<th></th>
<th>Monotonic</th>
<th>Cyclic</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Yu (2007)</strong></td>
<td>4x8x43x33-6-M-2</td>
<td>4x8x43x33-6-C-2</td>
</tr>
<tr>
<td></td>
<td>4x8x43x33-4-M-2</td>
<td>4x8x43x33-4-C-2</td>
</tr>
<tr>
<td></td>
<td>4x8x43x33-2-M-2</td>
<td>4x8x43x33-2-C-2</td>
</tr>
<tr>
<td></td>
<td>4x8x43x30-6-M-2</td>
<td>4x8x43x30-6-C-2</td>
</tr>
<tr>
<td></td>
<td>4x8x43x30-4-M-2</td>
<td>4x8x43x30-4-C-2</td>
</tr>
<tr>
<td></td>
<td>4x8x43x30-2-M-2</td>
<td>4x8x43x30-2-C-2</td>
</tr>
<tr>
<td></td>
<td>4x8x33x27-6-M-2</td>
<td>4x8x33x27-6-C-2</td>
</tr>
<tr>
<td></td>
<td>4x8x33x27-4-M-2</td>
<td>4x8x33x27-4-C-2</td>
</tr>
<tr>
<td></td>
<td>2x8x43x33-6-M-2</td>
<td>2x8x43x33-6-C-2</td>
</tr>
<tr>
<td></td>
<td>2x8x43x33-4-M-2</td>
<td>2x8x43x33-4-C-2</td>
</tr>
<tr>
<td></td>
<td>2x8x43x33-2-M-2</td>
<td>2x8x43x33-2-C-2</td>
</tr>
<tr>
<td></td>
<td>2x8x43x30-6-M-2</td>
<td>2x8x43x30-6-C-2</td>
</tr>
<tr>
<td></td>
<td>2x8x43x30-4-M-2</td>
<td>2x8x43x30-4-C-2</td>
</tr>
<tr>
<td></td>
<td>2x8x43x30-2-M-2</td>
<td>2x8x43x30-2-C-2</td>
</tr>
<tr>
<td><strong>Yu (2009)</strong></td>
<td>2x8x33x27-6-M-2</td>
<td>2x8x33x27-6-C-2</td>
</tr>
<tr>
<td></td>
<td>4x8x33x18-6-M-2</td>
<td>4x8x33x18-6-C-2</td>
</tr>
<tr>
<td></td>
<td>2x8x33x27-2-M-3</td>
<td>2x8x33x27-2-C-3</td>
</tr>
<tr>
<td></td>
<td>6x8x43x33-2-M-1-C</td>
<td>6x8x43x30-2-C-1-A</td>
</tr>
<tr>
<td></td>
<td>6x8x43x30-2-M-1-C</td>
<td>6x8x43x30-2-C-1-B</td>
</tr>
<tr>
<td></td>
<td>6x8x43x33-2-M-1-C</td>
<td>6x8x43x33-2-C-2-C</td>
</tr>
<tr>
<td></td>
<td>6x8x54x33-2-M-1-B</td>
<td>6x8x43x30-2-C-2-C</td>
</tr>
<tr>
<td></td>
<td>6x8x43x27-2-M-1-D</td>
<td>6x8x43x33-2-C-2-C</td>
</tr>
<tr>
<td></td>
<td>6x8x54x33-2-M-1-C</td>
<td>6x8x54x33-2-C-2-B</td>
</tr>
<tr>
<td><strong>Bahl (2010)</strong></td>
<td>4x8x43x18-6-M-3</td>
<td>6x8x43x27-2-C-1-D</td>
</tr>
<tr>
<td></td>
<td>4x8x43x18-2-M-2</td>
<td>6x8x54x33-2-C-2-C</td>
</tr>
<tr>
<td></td>
<td>4x8x33x18-6-M-2</td>
<td>4x8x43x33-2-C-2-C</td>
</tr>
<tr>
<td></td>
<td>4x8x43x30-6-M-2</td>
<td>2x8x43x33-2-C-2-C</td>
</tr>
<tr>
<td></td>
<td>4x8x43x30-4-M-2</td>
<td>4x8x43x18-6-C-2</td>
</tr>
<tr>
<td></td>
<td>4x8x43x30-2-M-2</td>
<td>4x8x43x18-2-C-2</td>
</tr>
<tr>
<td></td>
<td>2x8x43x30-4-M-2</td>
<td>4x8x33x18-6-C-2</td>
</tr>
<tr>
<td></td>
<td>2x8x43x30-2-M-3</td>
<td>4x8x43x30-6-C-2</td>
</tr>
<tr>
<td></td>
<td>2x8x33x30-4-M-1</td>
<td>4x8x43x30-4-C-2</td>
</tr>
<tr>
<td></td>
<td>8x8x43x30-4-M-2</td>
<td>4x8x43x30-2-C-2</td>
</tr>
<tr>
<td></td>
<td>6x8x43x30-4-M-1</td>
<td>2x8x43x30-4-C-2</td>
</tr>
<tr>
<td></td>
<td>6x8x43x30-2-M-1</td>
<td>2x8x43x30-2-C-2</td>
</tr>
<tr>
<td></td>
<td>4x8x43x30-4-M-1</td>
<td>8x8x43x30-4-C-2</td>
</tr>
<tr>
<td></td>
<td>4x8x43x18-2-M-2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4x8x43x18-3-M-1</td>
<td></td>
</tr>
</tbody>
</table>
Definition of test specimen label is presented in Table 3.2. Seismic detailing designation is only applicable to some of the specimens in Yu (2009). Full description and illustration of the seismic detailing of each specimen are available in Yu (2009).

![Table 3.2: Definition of test specimen label](image)

**Figure 3.11** Definition of test data label (steel sheet shear wall)

In Yu et al. (2007, 2009) and Balh (2010), material properties of test specimens were verified and reported. In this research, the development of the design formula of the effective strip width was based on actual measurements of the material thicknesses and mechanical properties. The measured material and mechanical properties of the tested specimens are presented in Table 3.3.
Table 3.3 Measured material properties (steel sheet shear wall)

<table>
<thead>
<tr>
<th>Test Program</th>
<th>Framing Member</th>
<th>Nominal Properties</th>
<th>Measured Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Thickness (in.)</td>
<td>Fy (ksi)</td>
</tr>
<tr>
<td>Yu (2007)</td>
<td>Steel Sheet</td>
<td>0.027</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.030</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.033</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td>Stud</td>
<td>0.033</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.043</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td>Track</td>
<td>0.033</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.043</td>
<td>33</td>
</tr>
<tr>
<td>Yu (2009)</td>
<td>Steel Sheet</td>
<td>0.018</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.027</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.030</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.033</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td>Stud</td>
<td>0.033</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.043</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.054</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td>Track</td>
<td>0.033</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.043</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.054</td>
<td>50</td>
</tr>
<tr>
<td>Balh (2010)</td>
<td>Steel Sheet</td>
<td>0.018</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.030</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td>Stud/Track</td>
<td>0.033</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.043</td>
<td>33</td>
</tr>
</tbody>
</table>

The proposed formula for the effective strip width is listed in Equation 3.14.

\[
W_e = \begin{cases} 
W_{max}, & \text{if } \lambda \leq 0.0819 \\
\rho W_{max}, & \text{if } \lambda > 0.0819 
\end{cases} 
\]

\[
\rho = \frac{1-0.55(\lambda-0.08)^{0.12}}{\lambda^{0.12}} 
\]

\[
\lambda = 1.736 \frac{\alpha_1 \alpha_2}{\beta_1 \beta_2 \beta_3 x^2 y} 
\]

where:

\[
W_{max} \quad \text{maximum width of effective strip as illustrated in Figure 3.12;}
\]

\[
= \frac{w}{\sin \alpha};
\]
\[ a = \text{aspect ratio of a shear wall (height / width)}; \]
\[ \alpha_1 = \frac{F_{ush}}{45}; \]
\[ \alpha_2 = \frac{F_{umin}}{45}; \]
\[ \beta_1 = \frac{t_{ush}}{0.018}; \]
\[ \beta_2 = \frac{t_{umin}}{0.018}; \]
\[ \beta_3 = \frac{s}{6}; \]
\[ F_{ush} = \text{tensile strength of steel sheet sheathing in ksi}; \]
\[ F_{umin} = \text{controlling tensile strength of framing materials in ksi (smaller tensile strength of track and stud)}; \]
\[ t_{ush} = \text{thickness of steel sheet sheathing in inches}; \]
\[ t_{umin} = \text{smaller of thicknesses of track and stud in inches}. \]

Figure 3.12 Maximum width of effective strip
Figure 3.13 shows a comparison between the proposed formulas of effective strip width with the experimental results. Based on the proposed effective strip model, $W_e$ can be obtained using Equation 3.17 for each test.

$$W_e = \max\left\{ \frac{2s(V_{test} \sin \alpha - P_{ns, tks} \sin \alpha \cos \alpha)}{P_{ns, t} \cos \alpha + P_{ns, s} \sin \alpha}, \frac{V_{test}}{tF_y \cos \alpha} \right\} \tag{3.17}$$

where $V_{test}$ is the peak load obtained from each shear wall test, and all the other notations are previously defined.

Figure 3.13 Comparison of proposed design curve with test results (actual material properties)

Figure 3.13 indicates that the proposed effective strip model and the design formula for the effective strip width work well for the CFS-SSSWs. The data points are well mixed since CFS-SSSWs demonstrated similar peak loads for monotonic and cyclic loading; therefore the proposed analytical model shall be used for both wind and seismic design. As mentioned earlier,
the proposed design equations to determine the effective strip width are developed by using actual material properties of the framing members. However, these properties are usually not readily accessible by design engineers. In order to allow the use of nominal material properties in the proposed design method, further analyses were carried out. Fig. 10 shows the comparison between the proposed design curve with test results when nominal mechanical properties and design thickness of steel sheathing and framing members are used to determine the effective strip width.

![Proposed Design Curve](image)

Figure 3.14 Comparison of proposed design curve with test results (nominal material properties)

When using nominal and design material properties, the proposed design equation is still able to capture the trend of the test data. Compared to Figure 3.13, in Figure 3.14, most of the test data points are slightly above the design curve. It indicates that when using nominal and
design material properties, the design equation yields a slightly conservative result compared to using the actual material properties. However the difference is not significant enough to require a different equation for $W_e$. The statistics of the comparisons is listed in Table 3.4. The results indicate that the use of nominal material properties in the proposed effective width method will yield 2.2% more conservative results with 15% greater variation than the results by using actual material properties. Those differences will be considered in the resistance factor calculation described in the following section.

Table 3.4 Statistical results of nominal shear strength values

<table>
<thead>
<tr>
<th>Material Property</th>
<th>No. of tests</th>
<th>$\frac{V_{Test}}{V_{Design}}$</th>
<th>Avg.</th>
<th>Std. dev.</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Actual</td>
<td>142</td>
<td>1.000</td>
<td>1.00</td>
<td>0.114</td>
<td>0.114</td>
</tr>
<tr>
<td>Nominal</td>
<td>142</td>
<td>1.022</td>
<td>1.02</td>
<td>0.133</td>
<td>0.131</td>
</tr>
</tbody>
</table>

During the development of the design formula, it was crucial to determine contributing factors or combinations of those factors that form trends in data plots so that design equations can be estimated based on those trends. Some of the contributing factors were obvious from test data (screw spacing and thickness of sheathing and framing members), but finding other factors and combinations of those factors was ultimately a try and error procedure. The ratio of effective strip width from test over maximum strip width (effective width ratio) was plot against a possible factor individually to see the trend of the plot. Then, the effective width ratio was plot against combinations of the factors which produced similar trends in order to increase the trend of the plot.

3.4 Discussion

The proposed effective strip model and design equations suggest that the effective strip width is controlled by the framing and sheathing’s thickness and tensile strength, fastener
spacing, and the wall’s aspect ratio. The proposed analytical model can be used to predict the shear capacity of the CFS-SSSWs without failures in boundary studs or hold-downs. The failures in boundary studs and hold-downs shall be successfully prevented if the designers follow the design guidance by AISI S213 (2007) which requires that the chord studs and uplift anchorage have the nominal strength to resist the lesser of the load that the system can deliver or the amplified seismic load.

The proposed design approach is developed based on actual thicknesses and mechanical properties of the test specimens. And it has been found that the actual mechanical properties of specimens are generally greater than the nominal or design values specified by the industry.

AISI S213 (2007) requires a reduction factor be used for CFS shear walls with an aspect ratio equal to or greater than 4 for the purpose of deflection control of slender walls. The proposed effective strip model determines the nominal strength without aspect ratio reduction for slender walls. Therefore the reduction factor in AISI S213 applies to the results calculated by the proposed design approach.

In order to verify the validity of the effective strip model and the design equation, published nominal shear strength values of CFS-SSSWs from Table C2.1-1 (wind) and Table C2.1-3 (seismic) in AISI S213 (2007) are used to compare with the nominal shear strength values determined based on the effective strip model and the proposed design equations. Table 3.5 shows the comparison of published nominal shear strength values with the values calculated by using effective strip model.
Table 3.5 Comparison of nominal shear strength values (effective strip model)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>2:1x33x18-6</td>
<td>485</td>
<td>390</td>
<td>399</td>
</tr>
<tr>
<td>4:1x43x27-4</td>
<td>1000</td>
<td>1000</td>
<td>785</td>
</tr>
<tr>
<td>4:1x43x27-3</td>
<td>1085</td>
<td>1085</td>
<td>891</td>
</tr>
<tr>
<td>4:1x43x27-2</td>
<td>1170</td>
<td>1170</td>
<td>1064</td>
</tr>
<tr>
<td>2:1x33x27-6</td>
<td>647</td>
<td>647</td>
<td>597</td>
</tr>
<tr>
<td>2:1x33x27-4</td>
<td>710</td>
<td>710</td>
<td>712</td>
</tr>
<tr>
<td>2:1x33x27-3</td>
<td>778</td>
<td>778</td>
<td>803</td>
</tr>
<tr>
<td>2:1x33x27-2</td>
<td>845</td>
<td>845</td>
<td>935</td>
</tr>
</tbody>
</table>

Table 3.5 is essentially equivalent to Table 3.1 except the predicted $V_n$ in Table 3.5 is replaced by calculated nominal shear strength values using effective strip model. The definition of wall configuration is also previously defined in Figure 3.7. The design parameters are same as the ones used in the comparison with strip model approach in section 3.1.3 the grade of steel sheet sheathing and framing members is considered to be ASTM A1003 Grade 33, having minimum yield strength of 33 ksi and tensile strength of 45 ksi. The sheathing-to-framing fastener size is #8 as specified in AISI S213 (2007). Nominal values are used for sheathing and framing material tensile strengths and screw diameters, and design values are used for sheathing and framing thicknesses to determine the nominal shear strength of each wall configuration.

According to the results shown in Table 3.5, most of the estimated nominal shear strength values are a little conservative or almost equivalent to the published values. Also, the developed analytical model is able to capture the trends of the impacts of key parameters (e.g. screw spacing, framing and sheathing material thickness, etc) to the shear wall strength.

A reliability analysis was also carried out to evaluate the proposed design approach by following the provisions in Chapter F of AISI S100 (2007). The resistance factors, $\phi$, for LRFD
approach can be determined in accordance with AISI S100 (2007) using a target reliability index, \( \beta \), of 2.5. The resistance factors, \( \phi \), can be determined as Equation 3.18.

\[
\phi = C_\phi (M_m F_m P_m) e^{-\beta \sqrt{V_M^2 + V_F^2 + C_P \beta^2 + V_Q^2}}
\]  

(3.18)

where:

\( C_\phi \) = calibration coefficient (1.52 for LRFD);
\( M_m \) = mean value of material factor (1.0 for actual, 1.178 for nominal);
\( F_m \) = mean value of fabrication factor (1.0 for actual, 0.965 for nominal);
\( P_m \) = mean value of professional factor (1.000 for actual, 1.022 for nominal);
\( e \) = natural logarithmic base (2.718);
\( \beta \) = target reliability index (2.5);
\( V_M \) = coefficient of variation of material factor (0.1 for actual, 0.085 for nominal);
\( V_F \) = coefficient of variation of fabrication factor (0.05 for actual, 0.053 for nominal);
\( C_P \) = correction factor (1.022);
\( V_P \) = coefficient of variation of test results (0.114 for actual, 0.131 for nominal);
\( V_Q \) = coefficient of variation of load factor (0.21 for LRFD).

For the case in which actual material properties (yield stress, tensile strength, thickness) are used (actual case), the values of \( M_m \), \( V_M \), \( F_m \), and \( V_F \) were taken from Table F1 in AISI S100 (2007) for “Structural Members Not Listed Above”. For the case in which nominal and design material properties were adopted (nominal case), the values of \( M_m \) and \( V_M \) were determined by taking the mean and the coefficient of variation of the ratios between the actual mechanical properties and the nominal mechanical properties of the materials used in the 142 specimens. Similarly, the factors regarding the effect of fabrication, \( F_m \) and \( V_F \) for the nominal case were
determined by taking the mean and the coefficient of variation of the ratio between the actual thickness and the design thickness of the materials used in the 142 specimens. According to the reliability analysis, the resistance factors are 0.79 for the actual case and 0.90 for the nominal case. Thus, it is recommended that the resistance factor of 0.79 be used when actual material properties are used in the design and the resistance factor of 0.90 be used when nominal and design material properties are used instead. The AISI S213 (2007) adopts an LRFD resistance factor of 0.65 for wind load design and 0.60 for seismic design.

ASD safety factors were also calculated for both actual and nominal cases by following the provisions in Chapter F of AISI S100 (2007) and using the following equation.

$$\Omega = \frac{1.6}{\phi} \quad (3.19)$$

where \(\Omega\) is the ASD safety factor and \(\phi\) is the LRFD resistance factor previously calculated in accordance with AISI S100 (2007). The safety factors for this method turned out to be 2.00 for the actual case and 1.80 for the nominal case. The AISI S213 (2007) adopts an ASD safety factor of 2.00 for wind load design and 2.50 for seismic design. The summary of the resistance factors and safety factors for this proposed design method is presented in Table 3.6.

Table 3.6 Summary of resistance factors and safety factors (effective strip model)

<table>
<thead>
<tr>
<th>Design Philosophy</th>
<th>Loading Type</th>
<th>AISI S213</th>
<th>Effective Strip Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>LRFD Resistance Factor ((\phi))</td>
<td>Wind</td>
<td>0.65</td>
<td>0.79 (Actual)</td>
</tr>
<tr>
<td></td>
<td>Seismic</td>
<td>0.60</td>
<td></td>
</tr>
<tr>
<td>ASD Safety Factor ((\Omega))</td>
<td>Wind</td>
<td>2.00</td>
<td>2.00 (Actual)</td>
</tr>
<tr>
<td></td>
<td>Seismic</td>
<td>2.50</td>
<td></td>
</tr>
</tbody>
</table>
The developed analytical model offers an accurate and reliable method to predict the nominal strength of CFS-SSSWs. The new approach provides designers an analytical way of determining the shear wall capacities without carrying out full-scale physical testing.

3.5 Summary

An analytical model – effective strip model is proposed in this thesis to predict the nominal strength of CFS-SSSWs. The proposed design approach shows consistent agreement with experimental results. The design equations are developed by using actual material and mechanical properties of the framing members. Further analyses indicate that the nominal material properties can also be used in the proposed equations to provide reliable shear wall strength. The resistance factors are calculated for both cases. The developed design equations provide designers an analytical tool to calculate the nominal strength of CFS-SSSWs without conducting full-scale shear wall tests.
CHAPTER 4
ANALYTICAL MODEL AND DESIGN METHOD FOR CFS SHEAR WALLS USING WOOD-BASED PANEL SHEATHING

4.1 Literature Review

Previously conducted experimental studies and analytical design approach of CFS framed wood-based panel shear walls (CFS-WPSWs) are reviewed in this section. Also, applicability of the proposed analytical model for CFS-SSSWs (effective strip model) to CFS-WPSWs is studied.

4.1.1 Previously Conducted Experimental Studies

Many researchers have conducted experimental investigations of CFS-WPSWs in the last 20 years. A testing program to investigate the behavior of CFS framed shear walls sheathed with plywood, OSB, and gypsum wallboard (GWB) was set by AISI’s shear wall task committee and performed by Serrette et al. (1996a, b) and Serrette (1997). This test program was divided in three phases. The primary purpose of phase 1 was to investigate the differences in static performance of plywood (15/32 in. American Plywood Association (APA) rated 4-ply) and OSB (7/16 in. APA rated) sheathed shear walls. In phase 2, the main objectives were to capture the characteristics of OSB sheathed walls with tighter fastener schedules, shear walls sheathed with OSB on one side and GWB on the other side, and shear walls with GWB on both sides under monotonic loading. The final phase of the test program focused on the cyclic testing of 4 ft. × 8 ft. OSB and plywood sheathed shear walls with different fastener spacing schedules.

The behavior of the OSB and plywood shear walls was found to be essentially identical for both monotonic and cyclic loading case. However, it was noted that the resistance of plywood shear walls was slightly greater in general. The dominant mode of failure was unzipping of the wall panel as a result of the panel being pulled over the screw heads. It was noted that the walls lost their load carrying capacity immediately after the screw pull-over failure.
Only three instances of screw pull-out from the stud flanges were observed in the entire test program.

Serrette (1997b) initiated a new test program in order to provide wider range of design options and verify some of the test results from Serrette (1996a, b) and Serrette (1997). The testing program was divided into five phases in which various materials were employed as sheathing: plywood, OSB, flat strap X-brace, and sheet steel. In phase 1, the main objective was to investigate the performance of 4 ft. × 8 ft. 15/32 in. plywood and 7/16 in. OSB shear walls framed with 0.033 in. studs. The sheathing was fastened to the framing with screws at 3 in. and 2 in. spacing. In phase 2, the main goal was to establish the limit on framing member thickness for sheathing attached with No. 8 screws. Same configuration and sheathing materials as phase 1 were employed except for the thickness of the studs (up to 0.054 in.). Phase 3 and 4 investigated the performance of shear walls with flat strap X-brace and sheet steel as sheathing materials. Finally, phase 5 investigated the behavior of shear walls with OSB and plywood with high aspect ratio (4:1).

With respect to CFS-WPSWs (OSB and plywood), Serrette (1997b) concluded that the wall assemblies with thicker framing members and tighter fastener spacing yielded greater shear resistance. The failure of the shear walls usually resulted from a combination of pull-over and pull-out of the screws. Also, Serrette et al. (1997b) noted that shear failure in screws was identified in the assemblies with 0.054 in. framing members.

In 2003, a test program of CFS-WPSWs was conducted at McGill University in Canada to assemble a large pool of data for different wall configurations constructed with Canadian steel and sheathing products, and the total of 109 shear wall specimens were tested under monotonic
and cyclic loading. The result of the test program was reported in Branston (2004), Chen (2004), and Boudreault (2005).

Branston (2004) performed 43 out of 109 tests. The test specimens were sheathed with 15/32 in. (12.5 mm) Douglas Fir Plywood (DFP) and Canadian Soft Plywood (CSP), as well as 7/16 in. (11 mm) performance rated OSB. Branston (2004) also reviewed existing data interpretation techniques and proposed an appropriate method for the analysis of all the data from the test program. The use of equivalent energy elastic-plastic (EEEP) method (originally developed by Park (1989) and further refined by Foliente (1996)) was recommended to determine the design parameters for in-plane stiffness and strength.

Chen (2004) conducted a total of 46 monotonic and cyclic testing of CFS framed shear walls sheathed with 15/32 in. CSP and 7/16 in. OSB. He investigated characteristics of the walls by evaluating the important design parameters by using the method recommended by Branston (2004). The design parameters included ultimate shear strength and yield strength, elastic stiffness, energy dissipation capacity and ductility, load capacity related to relative deflection, and steel chord capacity under compression. He also reviewed existing analytical models of wood framed wood–based panel sheathed shear walls and developed an analytical model of CFS framed wood-based panel sheathed shear walls to theoretically calculate the resistance and lateral deflection.

Boudreault (2005) reviewed existing reversed cyclic loading protocols for light framed shear walls in order to select the most appropriate protocol to be used in the test program. CUREE reversed cyclic loading protocol (Krawinkler et al., 2000; ASTM E2126, 2005) was selected to be incorporated in the test program and the remaining 20 tests of CFS shear walls with 15/32 in. DFP and CSP sheathing were carried out.
Branston (2004), Chen (2004), and Boudreault (2005) reported that the dominant mode of failure of the wall specimen involved CFS framing-to-sheathing screw connections. In most of the cases, bearing surface of the wood panel at screw connections were crushed by the screw heads, and the screws were pulled through the sheathing. Tear-out of the wood sheathing was another common failure, and usually occurred at panel edges and especially at the corners of the wall. A few instances of shear failure of the screws were identified especially at the corner locations of the walls. Surprisingly, no pull-out failure of the screws was identified in the entire test program.

In 2004, another test program of CFS-WPSW was conducted at McGill University to add more data for different wall configurations, and the total of 43 shear wall specimens were tested under monotonic and cyclic loading. The result of the test program was reported in Rokas (2006) and Blais (2006). In the research by Branston (2004), Chen (2004), and Boudreault (2005), the thickness of OSB sheathing was 7/16 in., and the thickness of DFP and CSP was 15/32. Rokas (2006) and Blais (2006) employed 3/8 in. OSB, DFP, and CSP. Blais (2006) tested 18 specimens with 3/8 in. OSB sheathing, and Rokas (2006) tested 25 specimens with 3/8 in. DFP and CSP sheathing under monotonic and cyclic loading.

In Rokas (2006) and Blais (2006), failure modes of the shear wall assemblies were very similar to the ones reported by Branston (2004), Chen (2004), and Boudreault (2005). Pull-over of screws and tear-out of the wood panel or the combination of those two were the most common cases. Shear failure in screws was also reported in a few instances.

In 2010, a test program of CFS-WPSWs was initiated and conducted by Dr. Cheng Yu and his students at University of North Texas to verify the published values of nominal strength provided in the tables C2.1-1 and C2.1-2 by AISI S213 (2007) and to propose shear strength
values of different wall configurations not listed in those tables. Li (2012) reported results of this test program for 36 shear walls sheathed with 7/16 in. OSB tested under both monotonic and cyclic loading protocols. In this test program, for OSB sheathed shear walls, in most cases, the bearing failure of the sheathing panel initiated the screws to be pulled though the sheathing. Also, sheathing bearing failure caused the edge tear-out, and shear failure of screws was also frequently observed. For some wall specimens with 2 in screw spacing, shear failure in the sheathing and buckling of chord studs were identified. Screw pull-out failure was rarely observed.

In the test program at UNT, 10 shear walls with plywood sheathing were also tested under monotonic and cyclic loading. The test results are included in the Appendix D of this thesis. These data will be incorporated in the development of the analytical model of CFS-WPSWs as well. For plywood sheathed shear walls, similar failures as OSB sheathed shear walls (pull-through, sheathing edge tear-out, and screw shear) were observed. Also, splits of the sheathing through the line of screw connections were identified in many cases regardless of the direction of the grain.

4.1.2 Related Analytical Models

As mentioned earlier, Chen (2004) reviewed existing analytical models of wood framed shear wall using wood-based panel sheathing. He reviewed several analytical models, but based on the assumption and applicability to CFS shear walls, he picked five models by Tuomi and McCutcheon (1978), Easley et al. (1982), and Kallsner and Lam (1995) to be further analyzed. These models are used to predict the lateral load capacity at yield shear strength level by using the EEEP method, and these values were compared with the shear resistance values recorded in his tests and the tests performed by Branston (2004) and Boudreault (2005).
Tuomi and McCutcheon (1978) developed an analytical model of wood framed shear walls to calculate the racking resistance of sheathing panels. The primary assumptions and limitations employed in this model are:

1. The entire external work is dissipated by the distortion of the fasteners at the sheathing-to-framing connections.
2. The lateral load versus displacement curve of single sheathing-to-framing connection is linear.
3. When lateral force is applied, the framing behaves as a parallelogram, but the sheathing stays rigid.
4. Spacing of the fasteners is even and in symmetry.
5. The speed of loading is slow enough so that the dynamic and impact effects are disregarded.
6. The fasteners at the four corners of the wall are free to rotate along the lines of the sheathing’s diagonal.

Easley et al. (1982) developed closed form equations to determine the strength of wood framed shear walls based on the deformation pattern of the wall specimens in actual testing. In this model, sheathing panels also become parallelogram along with the framing when lateral load is applied. The following is the assumptions and limitations of this model:

1. The forces experienced in the panel edges have horizontal and vertical components. The horizontal components of the forces are uniform, but the vertical components are proportional to the distance from the center of the panel.
2. The lateral load versus displacement curve of single sheathing-to-framing connection is linear.
3. Spacing of the fasteners is even and in symmetry

Kallsner and Lam (1995) developed three analytical models of wood framed shear walls in which, one is elastic and the other two are plastic models.

In the elastic model, the deformation and shear capacity of the walls were calculated based on the principal of minimum potential energy. The following assumptions and limitations are considered:

1. The framing members are rigid and hinged to each other.
2. The sheathing panel is rigid and free to rotate.
3. The lateral load versus displacement curve of sheathing-to-framing connection is linear.
4. Relative displacements between the framing and the sheathing are considered very small compared to the dimension of the wall.
5. The displacements of the center of the framing and the sheathing are the same, and there is no relative displacement between them.

In the first plastic model, lower bound model, the following assumptions and limitations are considered:

1. The lateral load versus displacement curve is completely plastic.
2. All the edge fasteners convey the same amount of force except for the ones at the corners of the wall.
3. Fasteners on top and bottom track carry only horizontal force, fasteners on boundary studs carry only vertical force, and fasteners at the corners of the wall carry half of both horizontal and vertical forces.
The second plastic model, upper bound model considers the following as assumption and limitations:

1. Each framing member has its own rotational center.

2. The framing members are hinged to each other.

3. All fasteners reach their plastic capacity at the same time.

4.1.3 Capacity of Individual Sheathing-to-Framing Connection

As indicated in the previous sections, it is essential to know the capacity of individual sheathing-to-framing connections for all the previously discussed analytical models in order to calculate the resistance of shear wall assemblies. Unlike, steel sheathing-to-steel framing screw connection capacity, calculation method of wood panel-to-steel framing connection capacities is not codified. Therefore, Okasha (2004) performed monotonic and cyclic tests of wood sheathing panel-to-steel framing connection by using the same sheathing and framing materials used in the test program by Branston (2004), Chen (2004), and Boudreault (2005).

The test specimen matrix was setup in a way that reflects all the connection configurations used in the full scale shear wall testing. The wood panel sheathing type included DFP, CSP, and OSB. The sheathing thickness included 3/8 in. and 15/32 in. DFP and CSP and 3/8 in. and 7/16 in. OSB. The framing thickness covered 0.033 in., 0.043 in., 0.054 in., and 0.068 in. Also, orientation of the loading with respect to the grain of the sheathing material and fastener edge distance were considered (0.24 in., 0.37 in., 0.5 in., 0.63 in., and 1.0 in.).

The monotonic tests were performed according to the ASTM Standard D1761 (1988), and the cyclic tests were performed in accordance with CUREE protocol. Two failure modes were reported. The first was screw pull-through from the sheathing, and the second was
sheathing tear-out at the edge. No screw pull-out failure was observed in the entire test program. It was also noted that the connection capacity of specimens loaded perpendicular to the grain of sheathing was consistently higher than the capacity of specimens loaded parallel to the grain. Also, based on the initial evaluation of monotonic test results, the shear resistance estimated with the connection test with 1.0 in. screw edge distance had greater match with the shear resistance values obtained from the full scale tests. Thus, the capacity of the connections of parallel-to-grain specimens with 1.0 in. screw edge distance was employed as the connection property to be utilized in the analysis of shear walls in Chen (2004). The results of the monotonic tests and cyclic tests by Okasha (2004) that were considered relevant to this research are listed in Table 4.1 and Table 4.2 respectively.

Table 4.1 Monotonic test results of sheathing-to-framing connection (Okasha, 2004)

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Max Load (lb)</th>
<th>EEEP Yield Load (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CSP</td>
<td>391</td>
<td>334</td>
</tr>
<tr>
<td>OSB</td>
<td>440</td>
<td>369</td>
</tr>
<tr>
<td>DFP</td>
<td>643</td>
<td>532</td>
</tr>
</tbody>
</table>

Table 4.2 Cyclic test result of sheathing to framing connection (Okasha, 2004)

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Max Load (lb)</th>
<th>EEEP Yield Load (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CSP</td>
<td>501</td>
<td>495</td>
</tr>
<tr>
<td>OSB</td>
<td>484</td>
<td>440</td>
</tr>
<tr>
<td>DFP</td>
<td>716</td>
<td>626</td>
</tr>
</tbody>
</table>

The shear resistance of the connections for cyclic tests is much higher than the one from monotonic test. Chen (2004) explained this with two reasoning. The first is the difference in loading protocol. The loading speed in cyclic tests is much faster compared to the monotonic loading. The second is the inconsistency in the sheathing material properties. It was noted that these two effects may have been more significant when only single connections were tested.
Thus, in order to achieve more rational and conservative prediction of the strength of the walls, Chen (2004) employed the results from monotonic connection tests to estimate the shear strength of walls for both monotonic and cyclic cases.

4.1.4 Chen (2004) – Simplified Strength Model (SSM)

By reviewing the assumptions, limitations, and formulation of the existing analytical models, Chen (2004) concluded that the analytical approach of wood framed wood panel shear walls can be applied to CFS framed wood-based panel shear walls. Chen (2004) adapted elastic model approach by Kallsner and Lam (2004) and simplified its assumptions and limitations in order to apply them to CFS framed shear walls with wood-based panel sheathing. The following is the list of assumptions formulated by Chen (2004) as the bases of a simplified analytical model, namely, simplified strength model.

1. Framing studs and tracks remain rigid and are hinged to each other.
2. Wood-based panels stay rigid throughout the loading process as well.
3. The lateral load versus displacement curves of the sheathing-to-framing connections are idealized as bilinear EEEP curves.
4. The relative displacements between the sheathing and framing are small compared to the entire shear wall dimension, and the wood sheathing and steel framing do not separate from each other throughout the loading process.
5. No relative displacement exists between the center of the sheathing panel and the corresponding location of the framing, so the origin of the established coordinate system on the panel and on the framing will be identical throughout the loading process.
6. Sheathing panel will not be joined to adjacent panels, meaning that the height of the sheathing corresponds to the height of the framing.

7. Rotation of the bottom track is prevented by full anchoring of the wall to the lower level support.

8. The entire external work done by the lateral loading is completely dissipated by the distortion of the sheathing-to-framing connection.

9. The sheathing-to-framing connection capacity is the same in all directions.

Figure 4.1 shows the relative displacement of the sheathing panel to the steel framing members based in the assumptions formulated by Chen (2004). The boundary studs rotate about the bottom base by the angle of $\gamma$, and the entire framing forms the parallelogram. Whereas, the sheathing panel remains rigid and rotates about its original center by the angle of $\varphi$. The resulting force distribution of sheathing-to-framing connections is presented in Figure 4.2.
Figure 4.1 Force distribution in frame members (Chen, 2004)

Figure 4.2 Force distribution in sheathing-to-framing connections (Chen, 2004)
Assuming that clockwise rotation is positive, the displacement of the framing relative to the sheathing at any point is given by the following equations. (Given that horizontal direction if X-axis and vertical direction is Y-axis)

Along X-axis: \[ u = u_{frame} - u_{panel} = (y - \varphi)y \] (4.1)
Along Y-axis: \[ v = v_{frame} - v_{panel} = \varphi x \] (4.2)

where:
- \( u \) = relative displacement of the framing to the sheathing panel in X direction;
- \( u_{frame} \) = displacement of the framing in the X direction;
- \( u_{panel} \) = displacement of the sheathing panel in the X direction;
- \( v \) = relative displacement of the framing to the sheathing panel in the Y direction;
- \( v_{frame} \) = displacement of the framing in the Y direction;
- \( v_{panel} \) = displacement of the sheathing panel in the Y direction;
- \( x \) = X coordinate of the location in concern relative to the original center of the sheathing panel;
- \( y \) = Y coordinate of the location in concern relative to the original center of the sheathing panel.

The force in each sheathing-to-framing connection can be broken into X and Y components and expressed as:

\[ S_{xi,conn} = k u_i = k(y - \varphi)y \] (4.3)
\[ S_{yi,conn} = k v_i = k \varphi y \] (4.4)

where:
- \( S_{xi,com} \) = X component of the force in sheathing-to-framing connections;
\( S_{yi,con}\) = Y component of the force in sheathing-to-framing connections;

\( i \) = the number of the fasteners;

\( k \) = the shear stiffness of sheathing-to-framing connections.

The sum of the potential energy based on the shear distortion of the entire sheathing-to-framing connections can be expressed as:

\[
U_1 = \sum_{i=1}^{N} \frac{1}{2} k (u_i^2 + v_i^2) \tag{4.5}
\]

where:

\( U_1 \) = the sum of the potential energy due to the shear distortion of the entire sheathing-to-framing connections;

\( N \) = the total number of the sheathing-to-framing connections.

The potential energy based on the displacement caused by the applied lateral load is expressed as:

\[
U_2 = -F \gamma H \tag{4.6}
\]

where:

\( F \) = the applied lateral load at the top of the shear wall;

\( H \) = the height of the shear wall.

By substituting Equations 4.3 and 4.4 into Equation 4.5 and summing the Equations 4.5 and 4.6, the total potential energy stored in the shear wall system can be determined by the following expression:

\[
U = U_1 + U_2 = \frac{1}{2} k \sum_{i=1}^{N} \left( [y_i^2 (\gamma - \varphi)^2] + x_i^2 \varphi^2 \right) - F \gamma H \tag{4.7}
\]

By taking partial derivatives of the above expression in terms of \( \gamma \) and \( \varphi \) and applying the principle of minimum potential energy, the resulting expressions are set to equal zero:
\[ \frac{\partial u}{\partial y} = 0 \quad \text{and} \quad \frac{\partial u}{\partial \varphi} = 0 \]  

(4.8)

Solving the above expressions, the resultant expressions are the following:

\[ k(y - \varphi) \sum_{i=1}^{N} y_i^2 - FH = 0 \]  

(4.9)

\[ k[-(y - \varphi) \sum_{i=1}^{N} y_i^2 + \varphi \sum_{i=1}^{N} x_i^2] = 0 \]  

(4.10)

Again, solving the above expressions and equating them with respect to \( y \) and \( \varphi \), the following expressions are derived:

\[ y = \frac{1}{k} FH \left( \frac{1}{\sum_{i=1}^{N} x_i^2} + \frac{1}{\sum_{i=1}^{N} y_i^2} \right) \]  

(4.11)

\[ \varphi = \frac{1}{k} FH \frac{1}{\sum_{i=1}^{N} x_i^2} \]  

(4.12)

Now, substituting Equations 4.11 and 4.12 into Equations 4.3 and 4.4, the each force component can be expressed without shear stiffness of sheathing-to-framing connections:

\[ S_{x_i,conn} = FH \frac{y_i}{\sum_{i=1}^{N} y_i^2} \]  

(4.13)

\[ S_{y_i,conn} = FH \frac{x_i}{\sum_{i=1}^{N} x_i^2} \]  

(4.14)

The resultant force of the sheathing-to-framing connections can be expressed as:

\[ S_{i,conn} = \sqrt{S_{x_i,conn}^2 + S_{y_i,conn}^2} = FH \sqrt{\left( \frac{x_i}{\sum_{i=1}^{N} x_i^2} \right)^2 + \left( \frac{y_i}{\sum_{i=1}^{N} y_i^2} \right)^2} \]  

(4.15)

where:

\[ S_{i,conn} = \text{the resultant force in the sheathing-to-framing connections.} \]
The maximum force is experienced at the farthest locations from the origin of the sheathing panel, which are the four corners of the wall panels, and the maximum force can be expressed as:

\[
S_{\text{max,conn}} = FH \sqrt{\left(\frac{x_{\text{max}}}{\sum_{i=1}^{N} x_i} \right)^2 + \left(\frac{y_{\text{max}}}{\sum_{i=1}^{N} y_i} \right)^2}
\] (4.16)

where:

- \(S_{\text{max,conn}}\) = the maximum force experienced in the sheathing-to-framing connections at the four corners of the wall;
- \(x_{\text{max}}\) = \(X\) coordinate of the sheathing-to-framing connections at the four corners of the shear walls;
- \(y_{\text{max}}\) = \(Y\) coordinate of the sheathing-to-framing connections at the four corners of the shear walls.

In simplified strength model, the failure of a shear wall assembly is determined when the maximum connection force reaches its design capacity, \(S_{y,\text{conn}}\), so the shear capacity of the entire wall assembly, \(S_{y,\text{wall}}\) can be expressed as:

\[
S_{y,\text{wall}} = \frac{S_{y,\text{conn}}}{H \sqrt{\left(\frac{x_{\text{max}}}{\sum_{i=1}^{N} x_i} \right)^2 + \left(\frac{y_{\text{max}}}{\sum_{i=1}^{N} y_i} \right)^2}}
\] (4.17)

Also, it is reasonable to assume that the ultimate capacity of a shear wall assembly, \(S_{u,\text{wall}}\) can be characterized by the ultimate capacity of sheathing-to-framing connections, \(S_{u,\text{conn}}\). Thus, the ultimate capacity of a shear wall can be expressed as:

\[
S_{u,\text{wall}} = \frac{S_{u,\text{conn}}}{H \sqrt{\left(\frac{x_{\text{max}}}{\sum_{i=1}^{N} x_i} \right)^2 + \left(\frac{y_{\text{max}}}{\sum_{i=1}^{N} y_i} \right)^2}}
\] (4.18)
It is worth noting that the shear capacity of a shear wall assembly is determined based on the wall height, configuration of the edge fasteners, and shear capacity of individual sheathing-to-framing connection. Equations 4.11 to 4.17 were originally presented by Kallsner and Lam (1995).

4.1.5 Chen (2004) – Comparison of Shear Wall Capacity

In order to verify the accuracy and reliability of Kallsner’s and Lam’s elastic model (simplified strength model), Chen (2004) performed comparisons between the shear capacity values from full scale testing and the values predicted by the analytical models. By using the connection test data (Table 4.1) by Okasha (2004), shear capacities of the walls were estimated at its yield capacity level and ultimate capacity level ($S_{y,wall}$ and $S_{u,wall}$ respectively). The estimated shear wall capacities were then compared to the shear capacity values obtained from Branston (2004), Chen (2004), and Boudreault (2005). The total of 32 wall configurations was considered in this comparison, which covered 103 out of 109 shear wall specimens. Also, the same comparison was conducted for the other analytical models including upper and lower bound models by Kallsner and Lam (1995) and models by Easley (1982) and Tuomi and McCutcheon (1978). Table 4.3 shows the result of the comparison including the ratio of full scale test value to estimated value and other statistical results.
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ratio</td>
<td>SD</td>
<td>COV</td>
<td>Ratio</td>
<td>SD</td>
<td>COV</td>
</tr>
<tr>
<td>Monotonic</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EEEP</td>
<td>1.050</td>
<td>0.122</td>
<td>0.116</td>
<td>1.022</td>
<td>0.120</td>
<td>0.117</td>
</tr>
<tr>
<td>Max Load</td>
<td>0.918</td>
<td>0.117</td>
<td>0.128</td>
<td>0.900</td>
<td>0.119</td>
<td>0.133</td>
</tr>
<tr>
<td>Cyclic</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>EEEP</td>
<td>1.012</td>
<td>0.129</td>
<td>0.127</td>
<td>0.985</td>
<td>0.125</td>
<td>0.126</td>
</tr>
<tr>
<td>Max Load</td>
<td>0.885</td>
<td>0.122</td>
<td>0.137</td>
<td>0.868</td>
<td>0.123</td>
<td>0.141</td>
</tr>
</tbody>
</table>
At EEEP yield capacity level, Kallsner’s and Lam’s elastic model, lower bound plastic model, and Tuomi’s and McCutcheon’s model show good agreements between the tested and the predicted. However, the elastic model shows slightly conservative ratios and is consistent in both monotonic and cyclic cases. The ratios show that the upper bound plastic model overestimates and Easley’s model underestimates the predicted values. Based on the statistical analysis results as well as the given assumptions and limitations of the model, Chen (2004) concluded that Kallsner’s and Lam’s elastic model was the best model to be used to predict the shear yield capacity of CFS-WPSWs. At the ultimate load level, most of the models overestimate the predicted values. Still, the elastic model gives fairly decent prediction at the ultimate load level among all the models compared.

Since one of the primary objectives of the research by Chen (2004) was to develop an analytical model of CFS-WPSWs to predict the shear resistance at its yield capacity, he did not give any conclusion about which model is suitable for predicting the shear wall lateral load capacity at its ultimate load level. However, combined with the statistical results by Chen (2004), the assumptions and limitations of the simplified strength model seem reasonable to be applied to CFS-WPSWs at their ultimate capacity level. Later in this research, the simplified strength model will be used as a base model to develop an analytical model for CFS-WPSWs.

4.2 Screw Connection Test – UNT Shear Wall Testing Program

As mentioned in the previous sections, the failures of CFS-WPSWs mainly occur at sheathing-to-framing connections. All the previously discussed analytical models require the capacity of individual sheathing-to-framing connections to be used to predict the shear capacity of the entire shear wall assembly. As noted earlier, Chen (2004) utilized the experimental data of connection tests done by Okasha (2004) to estimate the shear capacities of CFS-WPSWs of
different configurations and compared the results with the values obtained from the full scale tests by Branston (2004), Chen (2004), and Boudreault (2005). In Okasha (2004), the connection test specimens covered the entire connection configurations used in the tests.

Earlier in the literature review section of previously conducted full scale shear wall tests, a test program conducted at UNT was introduced. The data from this test program was later incorporated in this research for the development of an analytical model of CFS-WPSWs to estimate the shear resistance at their ultimate shear capacity. However, in order to do so, it was necessary to determine the shear capacity of individual sheathing-to-framing connections. Furthermore, it was important to cover all the connection types used in the test program at UNT.

In the connection tests done by Okasha (2004), cyclic loading case constantly resulted in greater capacity than monotonic loading case. In order to achieve more consistent prediction of full scale shear wall strength, Chen (2004) adapted only the monotonic connection test results and applied them to his analytical approach in both monotonic and cyclic loading cases. Thus, only monotonic connection tests were performed at UNT.

4.2.1 Test Setup

A total of 48 OSB and 24 plywood connection tests were performed under monotonic loading on 20 kip INSTRON® 4482 universal testing machine at UNT. The testing was conducted under displacement control mode with a loading rate of 0.3 in/min. The machine is connected to a PC, and during the testing, the machine provides digital reading of the applied force and displacement. Figure 4.3 shows the testing machine and its typical test setup with an OSB connection specimen installed.
The specimen is held to the bottom table by two clamps and is connected to the upper holder with five screws.

4.2.2 Test Specimen

For OSB connection tests, specimens were constructed with APA rated 7/16 in. OSB (24/16 span rating, exposure I) and 0.043 and 0.054 in. studs and tracks. For plywood connection tests, specimens were constructed with APA rated 15/32 in. 4-ply (32/16 span rating, structural I, exposure I) and 0.043 in. studs and tracks. Studs and tracks were placed web side down on the bottom table, and wood panels were set down flush to the bottom table and screwed to the stud or track at approximately half distance of the flange with No. 8 x 1.0 screws. The effect of the grain direction of sheathing panels was also considered in the development of test matrix. Figure 4.4 shows the close up of a plywood connection specimen from front and back. Also, Table 4.4 shows the test matrix of the entire connection test program.
Figure 4.4 Close-up of a connection test specimen

Table 4.4 Test matrix of sheathing-to-framing connection test

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Sheathing Type</th>
<th>Member Thickness (in.)</th>
<th>Orientation of Grain</th>
<th># of Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>43-S-OSB-V</td>
<td>APA rated 7/15 in. OSB</td>
<td>43 mil Stud</td>
<td>Vertical</td>
<td>6</td>
</tr>
<tr>
<td>43-S-OSB-H</td>
<td></td>
<td></td>
<td>Normal</td>
<td>6</td>
</tr>
<tr>
<td>43-T-OSB-V</td>
<td></td>
<td>43 mil Track</td>
<td>Vertical</td>
<td>6</td>
</tr>
<tr>
<td>43-T-OSB-H</td>
<td></td>
<td></td>
<td>Normal</td>
<td>6</td>
</tr>
<tr>
<td>54-S-OSB-V</td>
<td>APA rated 24/16 span rating</td>
<td>43 mil Stud</td>
<td>Vertical</td>
<td>6</td>
</tr>
<tr>
<td>54-S-OSB-H</td>
<td>Exposure I</td>
<td></td>
<td>Normal</td>
<td>6</td>
</tr>
<tr>
<td>54-T-OSB-V</td>
<td></td>
<td>54 mil Stud</td>
<td>Vertical</td>
<td>6</td>
</tr>
<tr>
<td>54-T-OSB-H</td>
<td></td>
<td></td>
<td>Normal</td>
<td>6</td>
</tr>
<tr>
<td>43-S-Ply-V</td>
<td>APA rated 15/32 in. 4-ply</td>
<td>43 mil Stud</td>
<td>Vertical</td>
<td>6</td>
</tr>
<tr>
<td>43-S-Ply-H</td>
<td>Structural I</td>
<td></td>
<td>Normal</td>
<td>6</td>
</tr>
<tr>
<td>43-T-Ply-V</td>
<td>APA rated 32/16 span rating</td>
<td>43 mil Track</td>
<td>Vertical</td>
<td>6</td>
</tr>
<tr>
<td>43-T-Ply-H</td>
<td>Exposure I</td>
<td></td>
<td>Normal</td>
<td>6</td>
</tr>
</tbody>
</table>
Each configuration was tested six times, and the average value of the ultimate shear capacities from the six identical tests was taken as the ultimate shear capacity of the given configuration. The definition of the test ID is presented in Figure 4.5.

![Figure 4.5 Definition of test ID (connection test)](image)

4.2.3 Test results

The result of the connection tests is summarized in Table 4.5. For each configuration, average value of the ultimate capacities recorded in the tests is presented in the second column from the left.

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Average Ultimate Shear Capacity (lb)</th>
<th>ESM Connection Shear Capacity (lb)</th>
<th>SSM Connection Shear Capacity (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>43-S-OSB-V</td>
<td>619</td>
<td>619</td>
<td>619</td>
</tr>
<tr>
<td>43-S-OSB-H</td>
<td>721</td>
<td>619</td>
<td></td>
</tr>
<tr>
<td>43-T-OSB-V</td>
<td>651</td>
<td>625</td>
<td></td>
</tr>
<tr>
<td>43-T-OSB-H</td>
<td>625</td>
<td></td>
<td></td>
</tr>
<tr>
<td>54-S-OSB-V</td>
<td>676</td>
<td>676</td>
<td>676</td>
</tr>
<tr>
<td>54-S-OSB-H</td>
<td>680</td>
<td>676</td>
<td></td>
</tr>
<tr>
<td>54-T-OSB-V</td>
<td>755</td>
<td>739</td>
<td></td>
</tr>
<tr>
<td>54-T-OSB-H</td>
<td>739</td>
<td></td>
<td></td>
</tr>
<tr>
<td>43-S-Ply-V</td>
<td>606</td>
<td>606</td>
<td>606</td>
</tr>
<tr>
<td>43-S-Ply-H</td>
<td>802</td>
<td></td>
<td></td>
</tr>
<tr>
<td>43-T-Ply-V</td>
<td>638</td>
<td>619</td>
<td></td>
</tr>
<tr>
<td>43-T-Ply-H</td>
<td>619</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

It is worth noting that the direction of the grain did not seem to affect the difference in the strength of the connections or at least there was no sign of such trends in this test program.
However, it was obvious that the connections with a thicker framing material yielded higher resistance consistently. These connection shear capacity values are later used along with previously discussed analytical models (effective strip model and simplified strength model) to estimate the ultimate shear capacity of CFS-WPSWs. The effective strip model requires three different types of connection capacities (connection with its screw connecting sheathing and stud, sheathing and track, and sheathing and both stud and track). The second column from the right in Table 4.5 shows the connection shear capacity values to be used in the effective strip model. The smaller of the shear capacity values between vertical and horizontal grains is used. The greater value between the connection capacity for the stud and track is used for the shear capacity of the connection that is penetrating both stud and track. The simplified strength model assumes that the shear capacity of sheathing-to-framing is same in all direction and also its difference is minimal between connections on track and stud. Therefore, the smaller value from the ESM connection shear capacity is employed for the values to be used in SSM as shown in the right column of Table 4.5.

The failure modes observed in these tests were fairly consistent with the ones identified in the full scale test program. For OSB connections, sheathing edge tear-out and shear failure of the screw was by far the most observed failure modes. Pull-through failure was rarely seen, but pull over failure was never observed in the test. For plywood connections, shear failure in screw and sheathing edge tear-out were also the dominant modes of failure. Also, split of the sheathing through the location of the screw was seen in some specimens.

4.3 Effective Strip Model

In chapter three, the effective strip model was developed to predict the nominal shear capacity of CFS-SSSWs. When a shear wall assembly is under lateral loading, the steel sheet
sheathing undergoes out-of-plane elastic buckling and create diagonal tension field. The effective strip model assumes that the diagonal tension field (effective strip) is responsible for carrying the entire tension force experienced in the sheathing. Thus, the ultimate shear capacity of a shear wall assembly is limited by the yield capacity of the effective strip or the capacity of sheathing-to-framing connections within the effective strip.

Mechanical properties between steel sheet sheathing and wood-based panels are completely different. Steel sheet sheathing is thin, flexible, and subject to out-of-plane buckling throughout the loading process. On the other hand, a wood-based panel stays rigid and shows minimal deformation across the panel. However, some of their failure modes are similar, and failures related to sheathing-to-framing connections are reported for both steel sheet and wood-based panel shear walls from previously discussed full scale testing programs. Besides, often times, sheathing-to-framing connection failures of wood-based panel shear walls are also identified at the corners of the walls, which could possibly be the region within its effective strip width. Therefore, in this section, assumptions and limitations of the effective strip model approach are employed to investigate if design equations of the effective strip width for CFS-WPSWs can be developed.

As discussed earlier in chapter three, in order to estimate the nominal shear strength of the shear walls, it is necessary to know the width of the effective strip. Similar method employed in chapter three will be used to develop the design equations to estimate the effective strip width. The total of 179 full scale shear wall test data (93 monotonic and 86 cyclic) from Branston (2004), Chen (2004), Boudreault (2005), Rokas (2006), Blais (2006), and the test program conducted at UNT (Li, 2012) are incorporated in this research to develop and verify the design equations. The test data from Serrette et al. (1996a, b, 1997a, and b) was not included in the
analysis since the sheathing-to-framing connection capacities of the tested specimens were not reported in these test programs. The list of test data used in this research is presented in Table 4.6.

Also, the definition of test specimen label is presented in Figure 4.6.

Table 4.6 List of test data label (wood-based panel shear wall)

<table>
<thead>
<tr>
<th>Monotonic</th>
<th>Cyclic</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>OSB</strong></td>
<td></td>
</tr>
<tr>
<td>UNT</td>
<td>8x2x43xOSB-2-M-2</td>
</tr>
<tr>
<td></td>
<td>8x2x43xOSB-6-M-2</td>
</tr>
<tr>
<td></td>
<td>8x2x54xOSB-2-M-2</td>
</tr>
<tr>
<td></td>
<td>8x4x43xOSB-2-M-2</td>
</tr>
<tr>
<td></td>
<td>8x4x43xOSB-6-M-2</td>
</tr>
<tr>
<td>Branston (2004)</td>
<td>8x4x43xOSB-6-M-3</td>
</tr>
<tr>
<td></td>
<td>8x4x43xOSB-4-M-3</td>
</tr>
<tr>
<td></td>
<td>8x4x43xOSB-3-M-3</td>
</tr>
<tr>
<td>Chen (2004)</td>
<td>8x2x43xOSB-6-M-3</td>
</tr>
<tr>
<td></td>
<td>8x2x43xOSB-4-M-3</td>
</tr>
<tr>
<td>Blais (2006)</td>
<td>8x4x43xOSB-6-M-3</td>
</tr>
<tr>
<td></td>
<td>8x4x43xOSB-4-M-3</td>
</tr>
<tr>
<td></td>
<td>8x4x43xOSB-3-M-3</td>
</tr>
<tr>
<td><strong>Plywood</strong></td>
<td></td>
</tr>
<tr>
<td>UNT</td>
<td>8x4x43xPLY-2-M-2</td>
</tr>
<tr>
<td></td>
<td>8x4x43xPLY-6-M-2</td>
</tr>
<tr>
<td>Chen (2004)</td>
<td>8x2x43xCSP-6-M-3</td>
</tr>
<tr>
<td></td>
<td>8x2x43xCSP-4-M-3</td>
</tr>
<tr>
<td></td>
<td>8x2x43xCSP-6-M-3</td>
</tr>
<tr>
<td></td>
<td>8x2x43xCSP-4-M-6</td>
</tr>
<tr>
<td></td>
<td>8x2x43xCSP-3-M-3</td>
</tr>
<tr>
<td>Boudreault (2004)</td>
<td>8x4x43xCSP-4-M-6</td>
</tr>
<tr>
<td></td>
<td>8x4x43xDFP-4-M-4</td>
</tr>
<tr>
<td>Branston (2004)</td>
<td>8x4x43xCSP-6-M-3</td>
</tr>
<tr>
<td></td>
<td>8x4x43xCSP-3-M-3</td>
</tr>
<tr>
<td></td>
<td>8x4x43xDFP-6-M-3</td>
</tr>
<tr>
<td></td>
<td>8x4x43xDFP-3-M-3</td>
</tr>
<tr>
<td>Rokas (2006)</td>
<td>8x4x43xCSP-6-M-6</td>
</tr>
<tr>
<td></td>
<td>8x4x43xCSP-4-M-6</td>
</tr>
<tr>
<td></td>
<td>8x4x43xCSP-3-M-3</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Each test program in Table 4.6 provided measured material properties of framing members used for the test specimens. The measured material and mechanical properties of the tested specimens are presented in Table 4.7.

Table 4.7 Measured material properties (wood-based panel shear wall)

<table>
<thead>
<tr>
<th>Test Program</th>
<th>Framing Member</th>
<th>Nominal Properties</th>
<th>Measured Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Thickness (in.)</td>
<td>Fy (ksi)</td>
<td>Thickness (in.)</td>
</tr>
<tr>
<td>UNT</td>
<td>Stud</td>
<td>0.043</td>
<td>33.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.054</td>
<td>50.0</td>
</tr>
<tr>
<td></td>
<td>Track</td>
<td>0.043</td>
<td>33.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.054</td>
<td>50.0</td>
</tr>
<tr>
<td>Branston et al.</td>
<td>Stud</td>
<td>0.043</td>
<td>33.0</td>
</tr>
<tr>
<td></td>
<td>Track</td>
<td>0.043</td>
<td>33.0</td>
</tr>
<tr>
<td>Rokas and Blais</td>
<td>Stud/Track</td>
<td>0.043</td>
<td>33.0</td>
</tr>
</tbody>
</table>

Figure 4.7 shows the plot of effective with to maximum effective width ratio (effective ratio) versus screw spacing. Figure 4.7 is just an example of one of the many plots investigated to see if there was any trend or correlation between the effective ratio and possible contributing factors or a combination of those factors. Unfortunately, it was unsuccessful to find any sort of trend or correlation of the effective ratio with respect to even a single factor.
Besides the fact that the analysis failed to find a trend of the plot of the effective ratio to estimate a design equation, it can be seen that the effective strip model approach is inherently not applicable to wood-based panel shear walls. In Figure 4.7, horizontal red line is drawn. This red line indicates the upper limit of the effective ratio. As previously noted, $W_{\text{max}}$ is the maximum effective strip width due to the geometrical limit of the wall configuration. The effective ratio of greater than 1.0 means that the contribution from the resistance of sheathing-to-framing connections within the effective strip was not enough to achieve the recorded peak load in the test. It indicates that the screws outside the maximum effective strip are also engaged in providing shear resistance. 73 out of 93 monotonic data points (78%), 48 out of 86 cyclic data points (56%), and 121 out of 179 total data points (68%) are above this red line. This result contradicts the important assumption of effective strip model that the ultimate shear capacity of a

![Figure 4.7 Effective ratio versus screw spacing](image)
shear wall is controlled by the capacity of sheathing-to-framing connections within the effective strip. This is also obvious when comparing Figure 4.7 to Figure 3.13. All the data points are below 1.0 in Figure 3.13.

This analysis indicates that the effective strip model is more than likely not applicable to wood-based panel shear walls. The reasoning for this difference between steel sheet and wood-based panel shear wall is that the characteristics of the sheathing materials probably dictate the behavior of the entire shear wall assembly. Thin and flexible steel sheet sheathing create diagonal tension field and exhibit obvious contribution of connections at the corners of the walls. However, this assumption is probably not applicable to wood-based panels. Wood-based panels are rigid and exhibit little deformation within the panel, thus the wood-based panels involve more screws around the parameter to resist the lateral loading.

4.4 Simplified Nominal Strength Model

Chen (2004) verified that the simplified strength model is capable of estimating the shear capacity of CFS-WPSWs at yield level by using EEEP method. However, he also noted that the simplified strength model overestimated the shear resistance of CFS-WPSWs at their ultimate capacity level. In this section, the foundations of the simplified strength model were employed, and its design equation was modified to predict the nominal shear strength of CFS-WPSWs more precisely. For convenience, the simplified strength model-based design equation to calculate the ultimate shear capacity of CFS-WPSWs is written as follows:

$$V_{ssn} = \frac{S_{u,conn}}{\sqrt{H \left( \frac{x_{max}}{\sum_{i=1}^{N} x_i} \right)^2 + \left( \frac{y_{max}}{\sum_{i=1}^{N} y_i} \right)^2}} \quad (4.19)$$

where:
\( V_{ssn} \) = ultimate shear capacity of CFS-WPSWs based on the simplified strength model approach.

The rest of the notations are same as previously defined.

The nominal shear strength of CFS-WPSWs is determined by multiplying the ultimate shear resistance from the simplified strength model approach by a modification factor to address the overestimation of the ultimate strength values. The design formula to calculate the nominal shear strength of CFS-WPSWs is expressed as follows:

\[
V_n = \rho V_{ssn} \tag{4.20}
\]

where:

\( V_n \) = nominal shear strength of CFS-WPSWs;

\( \rho \) = reduction factor;

\[
\rho = \begin{cases} 
0.202\alpha + 0.546, & \text{if } 1.0 \leq \alpha < 1.5 \\
0.068\alpha + 0.737, & \text{if } 1.5 \leq \alpha < 2.0 \\
0.873, & \text{if } 2.0 \leq \alpha \leq 3.0
\end{cases} \tag{4.21}
\]

\( \alpha = s/2; \)

\( s \) = parameter screw spacing in inches.

This design formula is also verified with 179 full scale shear wall test data from Branston (2004), Chen (2004), Boudreault (2005), Rokas (2006), Blais (2006), and test program conducted at UNT. Figure 4.8 shows the comparison between the propose design curve for the nominal shear strength and experimental data.
The identified trend in Figure 4.8 is that as the ratio gets lower as the spacing becomes narrower. This trend indicates that the overestimation of the ultimate shear resistance values is more significant when the screw spacing is closer. The data plots may look scattered, but most of the data points are around its average points where the design curve is going through. The data points of plywood are more scattered than those of OSB, but the average values of the ratio was very similar for both OSB and plywood at each location of the screw spacing. Also, both OSB and plywood sheathed shear walls performed similar under monotonic and cyclic loading and achieved similar peak load values. Thus, the proposed design approach shall be used for both OSB and plywood sheathed shear walls for both wind and seismic design. The statistical result of the ratio of the shear strength value from tests to the estimated value is presented in Table 4.8.
Table 4.8 Statistical results of nominal shear strength values

<table>
<thead>
<tr>
<th>No. of tests</th>
<th>$V_{Test}/V_n$</th>
<th>Avg.</th>
<th>Std. dev.</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>179</td>
<td></td>
<td>1.000</td>
<td>0.180</td>
<td>0.180</td>
</tr>
</tbody>
</table>

4.5 Discussion

The proposed simplified nominal strength model and its design equations are based on the assumptions and limitations of simplified strength model by Chen (2004). The obvious difference is that the simplified nominal strength model predicts the shear resistance of a CFS-WPSW at its ultimate capacity using the ultimate capacity of sheathing-to-framing connections. Whereas the simplified strength model predicts the shear yield capacity with using yield strength of sheathing-to-framing connections. Other than this difference, all the assumptions from the simplified strength model apply to the simplified nominal strength model.

Similar to the effective strip model for the CFS-SSSWs, requirements from AISI S213 (2007) also apply to the simplified nominal strength model, such as provisions to prevent failures in boundary studs and use of aspect ratio reduction factor for slender walls. More detailed discussion was provided in section 3.4.

In order to verify the validity of the simplified nominal strength model and the design equation, some of the published nominal shear strength values of CFS-WPSWs in AISI S213 (2007) are used to compare with the nominal shear strength values determined by the simplified nominal strength model. Table 4.9 shows the comparison of published nominal shear strength values with the values calculated by using simplified nominal strength model.
Table 4.9 Comparison of nominal shear strength values

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>2:1x43xPLY-6</td>
<td>1065</td>
<td>890</td>
<td>1105</td>
<td>713</td>
<td>1172</td>
</tr>
<tr>
<td>2:1x43xPLY-4</td>
<td>-</td>
<td>1330</td>
<td>1601</td>
<td>1033</td>
<td>1698</td>
</tr>
<tr>
<td>2:1x43xPLY-3</td>
<td>-</td>
<td>1775</td>
<td>2057</td>
<td>1300</td>
<td>2137</td>
</tr>
<tr>
<td>2:1x43xPLY-2</td>
<td>-</td>
<td>2190</td>
<td>2508</td>
<td>1618</td>
<td>2661</td>
</tr>
<tr>
<td>2:1x43xOSB-6</td>
<td>910</td>
<td>825</td>
<td>1129</td>
<td>802</td>
<td>802</td>
</tr>
<tr>
<td>2:1x43xOSB-4</td>
<td>1410</td>
<td>1235</td>
<td>1635</td>
<td>1162</td>
<td>1162</td>
</tr>
<tr>
<td>2:1x43xOSB-3</td>
<td>1735</td>
<td>1545</td>
<td>2014</td>
<td>1462</td>
<td>1462</td>
</tr>
<tr>
<td>2:1x43xOSB-2</td>
<td>1910</td>
<td>2060</td>
<td>2561</td>
<td>1821</td>
<td>1821</td>
</tr>
</tbody>
</table>

The published nominal shear strength values of CFS-WPSWs in AISI S213 (2007) are based on the test program by Serrette et al. (1996a, b and 1997). Since the ultimate capacity of individual sheathing-to-framing connections for the shear wall specimens used in these test programs was not reported, the nominal shear strength values were calculated by using the connection capacity values reported in the connection tests at UNT and the ones by Okasha (2004). Table 4.9 shows that the estimated values using connection test at UNT and DFP connection test values from Okasha (2004) are greater than the published values. Also, the estimated values using the CSP connection test values from Okasha (2004) are consistently less than the published values.

It should be noted that sheathing-to-framing connection capacity of CFS-WPSWs is inherent to the characteristics of the wood-based sheathing and the framing members actually used to construct the tested specimens. Thus, connection test data from one test program does not reflect the connection capacity characteristics of shear walls in other test programs. This is a major setback in this model compared to the effective strip model for CFS-SSSWs, in which, connection capacities can be determined by following a calculation method in a design
specification. However, the simplified nominal strength model captures the important trend of the shear strength values being higher as the screw spacing gets closer.

The reliability analysis was also performed for this model to evaluate its design approach by following the provisions in Chapter F of AISI S100 (2007). The details of the resistance factor calculation are previously introduced in section 3.4 of this thesis. The resistance factor can be calculated by using Equation 3.18. The parameters for the calculation of the resistance factor for this method are as follows.

\[
\begin{align*}
G_\phi &= 1.52 \text{ for LRFD;} \\
M_m &= 1.0; \\
F_m &= 1.0; \\
P_m &= 1.000; \\
e &= 2.718; \\
\beta &= 2.5; \\
V_M &= 0.1; \\
V_F &= 0.05; \\
G_p &= 1.017; \\
V_P &= 0.180; \\
V_Q &= 0.21 \text{ for LRFD.}
\end{align*}
\]

All of the above notations were previously defined in section 3.4, and the values of \(M_m, V_M, F_m, \) and \(V_F\) were taken from Table F1 in AISI S100 (2007) for “Structural Members Not Listed Above”. The AISI S213 (2007) uses a LRFD resistance factor of 0.65 for wind and 0.60 for seismic design. The resistance factor for the proposed design method turned out to be 0.72.
ASD safety factor was also calculated for this method by following the provisions in Chapter F of AISI S100 (2007) and using the Equation 3.19. The safety factors for this method turned out to be 2.20. The AISI S213 (2007) adopts an ASD safety factor of 2.00 for wind load design and 2.50 for seismic design. The summary of the resistance factor and safety factor for the proposed design method is presented in Table 4.10.

Table 4.10 Summary of resistance factor and safety factor (simplified nominal strength model)

<table>
<thead>
<tr>
<th>Design Philosophy</th>
<th>Loading Type</th>
<th>AISI S213</th>
<th>Simplified Nominal Strength Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>LRFD Resistance Factor (Φ)</td>
<td>Wind</td>
<td>0.65</td>
<td>0.72</td>
</tr>
<tr>
<td></td>
<td>Seismic</td>
<td>0.60</td>
<td></td>
</tr>
<tr>
<td>ASD Safety Factor (Ω)</td>
<td>Wind</td>
<td>2.00</td>
<td>2.20</td>
</tr>
<tr>
<td></td>
<td>Seismic</td>
<td>2.50</td>
<td></td>
</tr>
</tbody>
</table>

4.6 Summary

An analytical model – Simplified Nominal Strength Model is proposed in this thesis to predict the nominal strength of CFS-WPSWs. The proposed design approach shows consistent agreement with experimental results. Statistical analysis indicates the new method provides reliable results. This analytical method requires some efforts in testing of sheathing-to-framing connections to determine their ultimate shear capacity, but the developed design equations provide designers an analytical tool to calculate the nominal strength of CFS-WPSWs without conducting full-scale shear wall tests.
CHAPTER 5
CONCLUSIONS AND RECOMMENDATIONS

The primary objective of this research was to develop analytical models of CFS framed shear walls with steel sheet and wood-based sheathing. Existing analytical models from different structural systems were reviewed to seek an applicability of their assumptions, limitations, and its formulation of their design equations to estimate the nominal shear strength of shear wall assemblies to CFS shear wall systems. Previously conducted CFS shear wall testing programs were also reviewed to identify some of the most dominant failure mechanisms, and the test data from those shear wall testing programs were collected to develop and verify the analytical model and design equations. Finally, reliability analysis was conducted to determine the LRFD resistance factor and ASD safety factor for the proposed design approach.

5.1 Steel Sheet Shear Wall

An analytical model for CFS-SSSWs has not been developed from a comprehensive literature review. According to the literature review of previously conducted testing program for CFS-SSSWs, the most dominant failure mode was identified as the sheathing-to-framing screw connection failure within the observed diagonal tension field (effective strip). Based on this failure mechanism, an analytical model – effective strip model was developed to predict the nominal shear strength of CFS-SSSWs. 142 monotonic and cyclic full scale shear wall test data was collected and used to seek a consistent agreement between the proposed design approach and the experimental results. The design equations were developed by using actual material and mechanical properties of the sheathing and framing members. The proposed design approach show consistent agreement with the test results and also the AISI published nominal strength values of different wall configurations. Further analyses indicated that the nominal material
properties could also be used in the proposed equations to provide reliable shear wall strength. The resistance factors were calculated for both actual and nominal cases. The developed design equations provide designers an analytical tool to calculate the nominal strength of CFS-SSSWs without conducting full-scale shear wall tests.

5.2 Wood-based Panel Shear Wall

Based on the literature review of previously performed full scale testing of CFS-WPSWs, some of the most dominant failure mechanisms were sheathing edge tear-out, pull-through, and shear in connection screws or a combination of these failures. These failures are all related to a capacity of individual sheathing-to-framing connections. Along with the literature review of existing analytical models, it was found that the key to successfully estimate the nominal shear strength of CFS-WPSWs is finding the nominal capacity of sheathing-to-framing connections. Since the calculation method of the capacity of wood panel-to-steel framing screw connection is not codified in design provisions, its determination is solely dependent on experimental investigation. Thus, connection tests were conducted to determine the sheathing-to-framing connection capacity of shear wall specimens used in the full scale shear wall testing program at UNT. The effective strip model approach did not work for CFS-WPSWs because of the difference in its behavior and material properties. By applying the assumptions of the simplified strength model and elastic model, an analytical model – simplified nominal strength model was developed in this research to predict the nominal strength of CFS-WPSWs. The total of 179 monotonic and cyclic full scale shear wall test data was collected and used to seek an agreement between the proposed design approach and the experimental results. The proposed design approach show consistent agreement with the test results. The resistance factor was calculated for the LRFD design approach. This analytical method requires some efforts in testing of
sheathing-to-framing connections to determine their ultimate shear capacity, but if appropriate sheathing-to-framing connection capacities are provided, the developed design equations provide designers an analytical tool to calculate the nominal strength of CFS-WPSWs without conducting full-scale shear wall tests.

5.3 Recommendations for Future Research

In order to expand the knowledge about CFS-SSSWs and WPSWs and better understand the characteristics of their behaviors, many factors that are not incorporated in this research need to be addressed with respect to both testing and analytical models.

The configurations of the shear walls addressed in this research are very basic in terms of test specimens and wall configuration assumption of analytical models. However, in the real construction practice, CFS shear walls have more details such as ledger framing and opening in sheathing. Those details will more or less affect the performance of the shear walls. The influences of those detail need to be identified in full scale shear wall testing, and it is necessary to address in those influences analytical models and incorporate into a design procedure.

Especially in commercial building, along with the use of sheathing, steel ledger is frequently installed to the top of a framing to support floor joints. Steel ledgers are usually thicker than the framing members, and they are considered to add some stiffness and resistance to the shear wall system. Often times, the dimension of the shear walls with steel ledgers are 9ft. tall 4 ft. wide, which was not covered in this research.

Opening in sheathing is another example of the shear wall details often employed in actual construction. The openings are essential to install windows and doors. Testing of shear walls with perforated sheathing is necessary to capture the effect of the openings.
The combined effect of lateral loading and gravity loading has to be considered in the future research. In this research, shear walls are subjected to only in-plane lateral force. However, in a real situation, the structure and activities from the floors above the shear walls will exert gravity and live loading. The gravity loading may increase the stiffness of the structure and result in different load versus displacement relationship.

Most importantly, the connection test data has to be enhanced to obtain better estimation of the strength of individual sheathing-to-framing connections for CFS-WPSWs. In both effective strip model and simplified nominal strength model, it is crucial to obtain accurate sheathing-to-framing connection capacity for a better estimation of the nominal strength of an entire shear wall assembly. Unlike steel sheet, material properties of wood-based panels are not very consistent, and the determination of the connection capacity is totally dependent on testing. A better connection test procedure has to be developed to obtain more consistent results.
APPENDIX A

DESIGN EXAMPLE OF CFS-STEEL SHEET SHEAR WALL
A design example of 4 ft. × 8 ft. CFS-SSSW using 33 mil framing and 27 mil sheet steel sheathing is provided. The nominal yield stress of the framing and the sheathing are both 33 ksi. Sheathing-to-framing fasteners at the perimeter are No. 8 self-drilling tapping screws spaced at 3 inches. This design example assumes that actual material properties are obtained from coupon tests, and those actual material properties are used in calculation. The material properties of the framing members are listed in Table A.1:

Table A.1 Material properties of the framing members

<table>
<thead>
<tr>
<th>Member</th>
<th>Thickness (in.)</th>
<th>Yield Stress Fy (ksi)</th>
<th>Tensile Strength Fu (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>27 mil steel sheet</td>
<td>0.024</td>
<td>50.3</td>
<td>57.8</td>
</tr>
<tr>
<td>33 mil stud</td>
<td>0.033</td>
<td>47.7</td>
<td>55.7</td>
</tr>
<tr>
<td>33 mil track</td>
<td>0.033</td>
<td>57.4</td>
<td>67.2</td>
</tr>
</tbody>
</table>

Step 1: Estimating the effective strip width

\[ F_{ush} = 57.8 \text{ ksi} \]
\[ F_{uf} = 55.7 \text{ ksi} \]
\[ t_{sh} = 0.024 \text{ in.} \]
\[ t_f = 0.033 \text{ in.} \]
\[ a = (8 \text{ ft})/(4 \text{ ft}) = 2.0 \]
\[ \alpha = \tan^{-1} a = \tan^{-1}(2.0) = 63.4^\circ \]

Maximum effective width of the steel sheet sheathing

\[ W_{max} = \frac{W}{\sin \alpha} = \frac{W}{\sin(63.43^\circ)} = \frac{48 \text{ in.}}{\sin(63.43^\circ)} = 53.7 \text{ in.} \]
\[ \alpha_1 = F_{ush}/(45 \text{ ksi}) = (57.8 \text{ ksi})/(45 \text{ ksi}) = 1.284 \]
\[ \alpha_2 = F_{uf}/(45 \text{ ksi}) = (55.7 \text{ ksi})/(45 \text{ ksi}) = 1.238 \]
\[ \beta_1 = t_{sh}/(0.018 \text{ in.}) = (0.024 \text{ in.})/(0.018 \text{ in.}) = 1.333 \]
\[ \beta_2 = \frac{t_f}{(0.018 \text{ in.})} = \frac{0.033 \text{ in.}}{0.018 \text{ in.}} = 1.833 \]

\[ \beta_3 = \frac{s}{(6 \text{ in.})} = \frac{3 \text{ in.}}{6 \text{ in.}} = 0.5 \]

\[ \lambda = 1.736 \frac{\alpha_1 \alpha_2}{\beta_1 \beta_2 \beta_3^2 a} = 1.736 \times \frac{1.284 \times 1.238}{1.333 \times 1.833 \times (0.5)^2 \times 2.0} = 2.259 \]

\[ \lambda = 2.259 > 0.0819 \]

**Effective strip width of the steel sheet sheathing**

\[ W_e = \rho W_{max} = \frac{1 - 0.55(\lambda - 0.08)^{0.12}}{\lambda^{0.12}} W_{max} = \frac{1 - 0.55 \times (2.259 - 0.08)^{0.12}}{(2.259)^{0.12}} \times (53.7 \text{ in.}) \]

\[ W_e = 19.3 \text{ in.} \]

**Step 2: Determining the nominal shear capacity of individual connections**

**Sheathing-to-stud connection**

Connection shear limited by tilting and bearing

\[ t_1 = 0.024 \text{ in.} \]

\[ t_2 = 0.033 \text{ in.} \]

\[ d = 0.164 \text{ in.} \]

\[ F_{u1} = 57.8 \text{ ksi} \]

\[ F_{u2} = 55.7 \text{ ksi} \]

\[ \frac{t_2}{t_1} = \frac{0.033}{0.024} = 1.375 \]

\[ 1.0 < \frac{t_2}{t_1} < 2.5 \]

For \( \frac{t_2}{t_1} \leq 1.0, \)

\[ P_{ns} = \min \left\{ 4.2(t_2^3 d)^{1/2} F_{u2} = 4.2((0.033 \text{ in.})^3 \times (0.164 \text{ in.}))^{1/2} \times (55.7 \text{ ksi}) = 567.9 \text{ lb} \right. \]

\[ 2.7t_1 d F_{u1} = 2.7 \times (0.024 \text{ in.}) \times (0.164 \text{ in.}) \times (57.8 \text{ ksi}) = 614.3 \text{ lb} \]

\[ 2.7t_2 d F_{u2} = 2.7 \times (0.033 \text{ in.}) \times (0.164 \text{ in.}) \times (55.7 \text{ ksi}) = 813.9 \text{ lb} \]

For \( \frac{t_2}{t_1} \geq 2.5, \)
\[ P_{ns} = \min\left(2.7t_1dF_{u1} = 2.7 \times (0.024 \text{ in.}) \times (0.164 \text{ in.}) \times (57.8 \text{ ksi}) = 614.3 \text{ lb} \right. \\
\left. 2.7t_2dF_{u2} = 2.7 \times (0.033 \text{ in.}) \times (0.164 \text{ in.}) \times (55.7 \text{ ksi}) = 813.9 \text{ lb}\right] \\

By linear interpolating the smallest of the above two cases,

\[ P_{ns} = 578.7 \text{ lb} \]

Connection shear limited by end distance

\[ t = 0.024 \text{ in.} \]

\[ w_f = \text{flange width of stud} = 1.625 \text{ in} \]

\[ e = \frac{w_f}{2\cos\alpha} = \frac{1.625 \text{ in.}}{2 \times \cos 63.4^\circ} = 1.81 \text{ in.} \text{ (assuming that the screws are installed at the center of the flange of outer stud)} \]

\[ F_u = 57.8 \text{ ksi} \]

\[ P_{ns} = teF_u = (0.024 \text{ in.}) \times (1.81 \text{ in.}) \times (57.8 \text{ ksi}) = 2511 \text{ lb} \]

Connection shear limited by shear in screw

The nominal shear strength of screw is provided by the manufacturer. It is assumed that No. 8-18 Phillips Truss Head screw by HILTI is used. The screw shear strength can be found at a HILTI self-drilling screws report (ESR-2196, 2011).

\[ P_{ss} = 1170 \text{ lb} \]

\[ P_{ns,s} = \min\{579 \text{ lb, 2511 lb, 1170 lb}\} = 579 \text{ lb} \text{ (sheathing to stud connection)} \]

Similarly,

\[ P_{ns,t} = 614 \text{ lb} \text{ (sheathing to track connection)} \]

\[ P_{ns,t\&s} = 614 \text{ lb} \text{ (sheathing to track and stud connection at corners)} \]

Step 3: Determining the nominal shear strength of the shear wall

\[ t_{sh} = 0.024 \text{ in.} \]

\[ F_y = 50.3 \text{ ksi} \]
\[ V_n = \text{minimum} \left\{ \left( \frac{W_e}{2s \sin \alpha} P_{ns,t} + \frac{W_e}{2s \cos \alpha} P_{ns,s} + P_{ns,t&s} \right) \cos \alpha, W_e t F_y \cos \alpha \right\} \]

Limited by connection capacity

\[
\left( \frac{W_e}{2s \sin \alpha} P_{ns,t} + \frac{W_e}{2s \cos \alpha} P_{ns,s} + P_{ns,t&s} \right) \cos \alpha
= \left( \frac{19.3 \text{ in}}{2 \times (3 \text{ in}) \times \sin(63.43^\circ)} \times 614 \text{ lb} + \frac{19.3 \text{ in}}{2 \times (3 \text{ in}) \times \cos(63.43^\circ)} \times 579 \text{ lb} + 614 \text{ lb} \right) \times \cos(63.43^\circ) = 3125 \text{ lb}
\]

Limited by sheathing yield capacity

\[ W_e t_{sh} F_y \cos \alpha = (19.3 \text{ in.}) \times (0.024 \text{ in.}) \times (50.3 \text{ ksi}) \times \cos(63.43^\circ) = 10421 \text{ lb} \]

\[ V_n = \text{minimum} \{ 3125 \text{ lb}, 10421 \text{ lb} \} = 3125 \text{ lb} \]

**Nominal shear strength of the shear wall**

\[ V_n = 3125 \text{ lb} (781 \text{ lb/ft}) \]

The published value of nominal shear strength for this wall configuration in AISI S213-07 is 778 lb/ft, which is close to the calculated value by the proposed method.

**LRFD design shear strength of the shear wall**

\[ \phi V_n = 0.79 \times (781 \text{ lb/ft}) = 617 \text{ lb/ft} \]
APPENDIX B

DESIGN EXAMPLE OF CFS-WOOD-BASED PANEL SHEAR WALL
A design example of 4 ft \times 8 \text{ ft} CFS-WPSW using 43 mil framing and 7/16 in. OSB sheathing is provided. The nominal yield stress of the framing and the sheathing are both 33 ksi. Sheathing-to-framing fasteners at the perimeter are No. 8 self-drilling tapping screws spaced at 6 inches. This design example uses the connection test data from UNT in calculation.

**Capacity of individual OSB sheathing-to-framing connections**

\[ S_{u,\text{conn}} = 619 \text{ lb} \]

The coordinate of the farthest screw from the origin (origin center of the sheathing) is the screw at the four corner of the wall.

\[(x_{\text{max}}, y_{\text{max}}) = (23 \text{ in.}, 47 \text{ in.})\]

Sum of square of each horizontal coordinate of the screws

\[ \sum_{i=1}^{N} x_i^2 = 22068 \text{ in.}^2 \]

Sum of square of each vertical coordinate of the screws

\[ \sum_{i=1}^{N} y_i^2 = 68660 \text{ in.}^2 \]

The shear resistance of the wall at its ultimate capacity level based on the simplified strength model

\[ V_{ssn} = \frac{S_{u,\text{conn}}}{H \sqrt{\left(\frac{x_{\text{max}}}{\sum_{i=1}^{N} x_i^2}\right)^2 + \left(\frac{y_{\text{max}}}{\sum_{i=1}^{N} y_i^2}\right)^2}} = \frac{619 \text{ lb}}{96 \text{ in.} \times \sqrt{\left(\frac{23 \text{ in.}}{22068 \text{ in.}^2}\right)^2 + \left(\frac{47 \text{ in.}}{68660 \text{ in.}^2}\right)^2}} = 5171 \text{ lb} \]

\[ V_{ssn} = 5171 \text{ lb} = 1293 \text{ lb/ft} \]

Determine the reduction factor based on the spacing of the sheathing-to-framing fasteners

\[ s = 6 \text{ in.} \quad (4 \text{ in.} \leq s \leq 6 \text{ in.}) \]

\[ \rho = 0.873 \]

**Nominal shear strength of the shear wall**
\[ V_n = \rho V_{ssn} = 0.873 \times (1293 \text{ lb/ft}) = 1129 \text{ lb/ft} \]

*LRFD design shear strength of the shear wall*

\[ \phi V_n = 0.72 \times (1129 \text{ lb/ft}) = 813 \text{ lb/ft} \]
APPENDIX C

DATA SHEETS OF OSB AND PLYWOOD CONNECTION TESTS
Test Label: osb_43_stud_horz_#1

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Stud: 33 ksi 362S162-43
Grain direction: Horizontal
Fastener: #8-18x1” modified truss head self-drilling screw

Test results
Maximum load: 774.2 lbs
Net lateral displacement at top of wall at Maximum load: 0.453 in.

Observed Failure Mode: Sheathing panel bearing failure/sheathing edge tear-out in a wide range
Test Label: osb_43_stud_horz_#2

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Stud: 33 ksi, 362S162-43
Grain direction: Horizontal
Fastener: #8-18x1” modified truss head self-drilling screw

Test results
Maximum load: 722.2 lbs
Net lateral displacement at top of wall at Maximum load: 0.434 in.

Observed Failure Mode: Shear failure in the screw
Test Label: osb_43_stud_horz_#3

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Fastener: #8-18x1” modified truss head self-drilling screw

Stud: 33 ksi, 362S162-43
Grain direction: Horizontal

Test results
Maximum load: 748.5 lbs
Net lateral displacement at top of wall at Maximum load: 0.532 in.

Observed Failure Mode: Sheathing edge tear-out/bearing failure
Test Label: osb_43_stud_horz_#4

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Fastener: #8-18x1” modified truss head self-drilling screw

Stud: 33 ksi, 362S162-43
Grain direction: Horizontal

Test results
Maximum load: 740.4 lbs
Net lateral displacement at top of wall at Maximum load: 0.354 in.

Observed Failure Mode: Shear failure in the screw
Test Label: osb_43_stud_horz_#5

Specimen Configuration
- Wall dimension: 4 1/2 in. × 6 1/2 in.
- Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
- Fastener: #8-18x1” modified truss head self-drilling screw

Stud: 33 ksi, 362S162-43
Grain direction: Horizontal

Test results
- Maximum load: 699.6 lbs
- Net lateral displacement at top of wall at Maximum load: 0.392 in.

Observed Failure Mode: Shear failure in the screw
Test Label: osb_43_stud_horz_#6

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.  
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB  
Stud: 33 ksi, 362S162-43  
Fastener: #8-18x1” modified truss head self-drilling screw  
Grain direction: Horizontal

Test results
Maximum load: 644.3 lbs  
Net lateral displacement at top of wall at Maximum load: 0.422 in.

Observed Failure Mode: Sheathing edge tear-out/bearing failure
Test Label: osb_43_stud_vert_#1

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Fastener: #8-18x1” modified truss head self-drilling screw

Stud: 33 ksi, 362S162-43
Grain direction: Vertical

Test results
Maximum load: 620.7 lbs
Net lateral displacement at top of wall at Maximum load: 0.363 in.

Observed Failure Mode: Sheathing edge tear-out/bearing failure
**Test Label: osb 43 stud vert #2**

**Specimen Configuration**
- Wall dimension: 4 1/2 in. × 6 1/2 in.
- Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
- Fastener: #8-18x1” modified truss head self-drilling screw

**Stud:** 33 ksi, 362S162-43
**Grain direction:** Vertical

**Test results**
- Maximum load: 548.2 lbs
- Net lateral displacement at top of wall at Maximum load: 0.348 in.

**Observed Failure Mode:** Sheathing edge tear-out/bearing failure
Test Label: osb_43_stud_vert_#3

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.  
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB  
Stud: 33 ksi, 362S162-43  
Fastener: #8-18x1” modified truss head self-drilling screw  
Grain direction: Vertical

Test results
Maximum load: 636.2 lbs  
Net lateral displacement at top of wall at Maximum load: 0.318 in.

Observed Failure Mode: Sheathing edge tear-out/bearing failure
Test Label: osb_43_stud_vert_#4

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Fastener: #8-18x1” modified truss head self-drilling screw

Stud: 33 ksi, 362S162-43
Grain direction: Vertical

Test results
Maximum load: 736.1 lbs
Net lateral displacement at top of wall at Maximum load: 0.466 in.

Observed Failure Mode: Sheathing edge tear-out/bearing failure
Test Label: osb_43_stud_vert_#5

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.  
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB  
Fastener: #8-18x1” modified truss head self-drilling screw  
Stud: 33 ksi, 362S162-43  
Grain direction: Vertical

Test results
Maximum load: 553.0 lbs  
Net lateral displacement at top of wall at Maximum load: 0.355 in.

Observed Failure Mode: Sheathing edge tear-out/bearing failure
Test Label: osb_43_stud_vert_#6

Specimen Configuration
Wall dimension: 4 1/2 in. x 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Fastener: #8-18x1” modified truss head self-drilling screw

Stud: 33 ksi, 362S162-43
Grain direction: Vertical

Test results
Maximum load: 619.6 lbs
Net lateral displacement at top of wall at Maximum load: 0.233 in.

Observed Failure Mode: Shear failure in the screw
Test Label: osb_43_track_horz_#1

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Fastener: #8-18x1” modified truss head self-drilling screw

Track: 33ksi, 362T150-43
Grain direction: Horizontal

Test results
Maximum load: 666.3 lbs
Net lateral displacement at top of wall at Maximum load: 0.253 in.

Observed Failure Mode: Shear failure in the screw
Test Label: osb_43_track_horz_#2

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Fastener: #8-18x1” modified truss head self-drilling screw

Track: 33ksi, 362T150-43
Grain direction: Horizontal

Test results
Maximum load: 652.9 lbs
Net lateral displacement at top of wall at Maximum load: 0.265 in.

Observed Failure Mode: Shear failure in the screw
Test Label: **osb_43_track_horz_#3**

**Specimen Configuration**
- Wall dimension: 4 1/2 in. × 6 1/2 in.
- Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
- Fastener: #8-18x1” modified truss head self-drilling screw

**Track**: 33ksi, 362T150-43  
**Grain direction**: Horizontal

**Test results**
- Maximum load: **620.1 lbs**
- Net lateral displacement at top of wall at Maximum load: **0.29 in.**

**Observed Failure Mode**: Sheathing edge tear-out/bearing failure
Test Label: osb_43_track_horz_#4

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Fastener: #8-18x1” modified truss head self-drilling screw

Track: 33ksi, 362T150-43
Grain direction: Horizontal

Test results
Maximum load: 541.8 lbs
Net lateral displacement at top of wall at Maximum load: 0.380 in.

Observed Failure Mode: Sheathing tear-out/bearing failure
Test Label: osb_43_track_horz_#5

Specimen Configuration
Wall dimension: 4 1/2 in. x 6 1/2 in.  
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB  
Fastener: #8-18x1” modified truss head self-drilling screw

Track: 33ksi, 362T150-43  
Grain direction: Horizontal

Test results
Maximum load: 664.2 lbs  
Net lateral displacement at top of wall at Maximum load: 0.373 in.

Observed Failure Mode: Sheathing tear-out/bearing failure
Test Label: osb_43_track_horz_#6

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Fastener: #8-18x1” modified truss head self-drilling screw

Track: 33ksi, 362T150-43
Grain direction: Horizontal

Test results
Maximum load: 604.0 lbs
Net lateral displacement at top of wall at Maximum load: 0.320 in.

Observed Failure Mode: Sheathing tear-out/bearing failure
Test Label: osb_43_track_vert #1

Specimen Configuration
Wall dimension: 4 1/2 in. x 6 1/2 in.  
Track: 33ksi, 362T150-43  
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB  
Grain direction: Vertical  
Fastener: #8-18x1” modified truss head self-drilling screw

Test results
Maximum load: 426.9 lbs  
Net lateral displacement at top of wall at Maximum load: 0.218 in.

Observed Failure Mode: Sheathing edge tear-out/bearing failure
Test Label: osb_43_track_vert #2

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Fastener: #8-18x1” modified truss head self-drilling screw

Track: 33ksi, 362T150-43
Grain direction: Vertical

Test results
Maximum load: 516.0 lbs
Net lateral displacement at top of wall at Maximum load: 0.332 in.

Observed Failure Mode: Sheathing edge tear-out/bearing failure
Test Label: osb_43_track_vert #3

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.  Track: 33ksi, 362T150-43
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB  Grain direction: Vertical
Fastener: #8-18x1” modified truss head self-drilling screw

Test results
Maximum load: 608.3 lbs
Net lateral displacement at top of wall at Maximum load: 0.35 in.

Observed Failure Mode: Sheathing edge tear-out/bearing failure
Test Label: osb_43_track_vert #4

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Fastener: #8-18x1” modified truss head self-drilling screw

Test results
Maximum load: 778.0 lbs
Net lateral displacement at top of wall at Maximum load: 0.268 in.

Observed Failure Mode: Shear failure in the screw
Test Label: osb_43_track_vert #5

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Fastener: #8-18x1” modified truss head self-drilling screw
Track: 33ksi, 362T150-43
Grain direction: Vertical

Test results
Maximum load: 768.3 lbs
Net lateral displacement at top of wall at Maximum load: 0.262 in.

Observed Failure Mode: Shear failure in the screw
Test Label: osb 43_track_vert #6

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Fastener: #8-18x1” modified truss head self-drilling screw

Track: 33ksi, 362T150-43
Grain direction: Vertical

Test results
Maximum load: 808.1 lbs
Net lateral displacement at top of wall at Maximum load: 0.491 in.

Observed Failure Mode: Sheathing edge tear-out/bearing failure
Test Label: osb_54_stud_horz_#1

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Stud: 50ksi, 362S162-54
Fastener: #8-18x1” modified truss head self-drilling screw
Grain direction: Horizontal

Test results
Maximum load: 850.5 lbs
Net lateral displacement at top of wall at Maximum load: 0.429 in.

Observed Failure Mode: Sheathing edge tear-out/bearing failure
Test Label: osb_54_stud_horz_#2

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Stud: 50ksi, 362S162-54
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Grain direction: Horizontal
Fastener: #8-18x1” modified truss head self-drilling screw

Test results
Maximum load: 728.6 lbs
Net lateral displacement at top of wall at Maximum load: 0.356 in.

Observed Failure Mode: Shear failure in the screw
Test Label: osb_54_stud_horz_#3

Specimen Configuration
Wall dimension: 4 1/2 in. x 6 1/2 in.  Stud: 50ksi, 362S162-54
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB  Grain direction: Horizontal
Fastener: #8-18x1” modified truss head self-drilling screw

Test results
Maximum load: 685.1 lbs
Net lateral displacement at top of wall at Maximum load: 0.434 in.

Observed Failure Mode: Sheathing edge tear-out/bearing failure
Test Label: osb_54_stud_horz_#4

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Fastener: #8-18x1” modified truss head self-drilling screw
Stud: 50ksi, 362S162-54
Grain direction: Horizontal

Test results
Maximum load: 514.4 lbs
Net lateral displacement at top of wall at Maximum load: 0.442 in.

Observed Failure Mode: Sheathing edge tear-out/bearing failure
Test Label: osb_54_stud_horz_#5

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Fastener: #8-18x1” modified truss head self-drilling screw

Stud: 50ksi, 362S162-54
Grain direction: Horizontal

Test results
Maximum load: 626.6 lbs
Net lateral displacement at top of wall at Maximum load: 0.469 in.

Observed Failure Mode: Sheathing tear along the grain at the location of the screw/bearing failure
Test Label: osb_54_stud_horz_#6

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.  Stud: 50ksi, 362S162-54
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB  Grain direction: Horizontal
Fastener: #8-18x1” modified truss head self-drilling screw

Test results
Maximum load: 673.3 lbs
Net lateral displacement at top of wall at Maximum load: 0.337 in.

Observed Failure Mode: Sheathing tear-out/bearing failure
Test Label: osb_54_stud_vert_#1

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.  Stud: 50ksi, 362S162-54
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB  Grain direction: Vertical
Fastener: #8-18x1” modified truss head self-drilling screw

Test results
Maximum load: 680.8 lbs
Net lateral displacement at top of wall at Maximum load: 0.276 in.

Observed Failure Mode: Shear failure in the screw
Test Label: osb_54_stud_vert_#2

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in. Stud: 50ksi, 362S162-54
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB Grain direction: Vertical
Fastener: #8-18x1” modified truss head self-drilling screw

Test results
Maximum load: 583.6 lbs
Net lateral displacement at top of wall at Maximum load: 0.258 in.

Observed Failure Mode: Shear failure in the screw
Test Label: osb_54_stud_vert_#3

Specimen Configuration
Wall dimension: 4 1/2 in. x 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Stud: 50ksi, 362S162-54
Fastener: #8-18x1” modified truss head self-drilling screw
Grain direction: Vertical

Test results
Maximum load: 606.7 lbs
Net lateral displacement at top of wall at Maximum load: 0.271 in.

Observed Failure Mode: Shear failure in the screw
Test Label: osb_54_stud_vert_#4

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.  
Stud: 50ksi, 362S162-54  
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB  
Fastener: #8-18x1” modified truss head self-drilling screw  
Grain direction: Vertical

Test results
Maximum load: 751.1 lbs  
Net lateral displacement at top of wall at Maximum load: 0.301 in.

Observed Failure Mode: Shear failure in the screw
Test Label: osb_54_stud_vert_#5

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Stud: 50ksi, 362S162-54
Fastener: #8-18x1” modified truss head self-drilling screw
Grain direction: Vertical

Test results
Maximum load: 677.6 lbs
Net lateral displacement at top of wall at Maximum load: 0.284 in.

Observed Failure Mode: Shear failure in the screw
Test Label: osb_54_stud_vert_#6

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Fastener: #8-18x1” modified truss head self-drilling screw
Stud: 50ksi, 362S162-54
Grain direction: Vertical

Test results
Maximum load: 756.0 lbs
Net lateral displacement at top of wall at Maximum load: 0.293 in.

Observed Failure Mode: Shear failure in the screw
Test Label: osb_54_track_horz_#1

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.  Track: 50ksi, 362S150-54
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB  Grain direction: Horizontal
Fastener: #8-18x1” modified truss head self-drilling screw

Test results
Maximum load: 617.5 lbs
Net lateral displacement at top of wall at Maximum load: 0.341 in.

Observed Failure Mode: Shear failure in the screw
**Test Label: osb_54_track_horz_#2**

**Specimen Configuration**
- Wall dimension: 4 1/2 in. × 6 1/2 in.
- Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
- Fastener: #8-18x1” modified truss head self-drilling screw

**Test results**
- Maximum load: 891.8 lbs
- Net lateral displacement at top of wall at Maximum load: 0.396 in.

**Observed Failure Mode:** Sheathing edge tear-out/bearing failure
Test Label: osb_54_track_horz_#3

Specimen Configuration
Wall dimension: 4 1/2 in. x 6 1/2 in.  Track: 50ksi, 362S150-54
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB  Grain direction: Horizontal
Fastener: #8-18x1” modified truss head self-drilling screw

Test results
Maximum load: 638.9 lbs
Net lateral displacement at top of wall at Maximum load: 0.382 in.

Observed Failure Mode: Sheathing edge tear-out/bearing failure
Test Label: osb_54_track_horz_#4

Specimen Configuration
Wall dimension: 4 1/2 in. x 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Fastener: #8-18x1” modified truss head self-drilling screw

Track: 50ksi, 362S150-54
Grain direction: Horizontal

Test results
Maximum load: 789.3 lbs
Net lateral displacement at top of wall at Maximum load: 0.379 in.

Observed Failure Mode: Sheathing tear along the grain through the location of the screw
Test Label: osb_54_track_horz_#5

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.  
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB  
Fastener: #8-18x1” modified truss head self-drilling screw  
Track: 50ksi, 362S150-54  
Grain direction: Horizontal

Test results
Maximum load: 808.6 lbs  
Net lateral displacement at top of wall at Maximum load: 0.319 in.

Observed Failure Mode: Sheathing tear-out/bearing failure
Test Label: osb_54_track_horz_#6

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Fastener: #8-18x1” modified truss head self-drilling screw

Track: 50ksi, 362S150-54
Grain direction: Horizontal

Test results
Maximum load: 686.2 lbs
Net lateral displacement at top of wall at Maximum load: 0.516 in.

Observed Failure Mode: Sheathing bearing failure/pull-through
Test Label: **osb_54_track_vert #1**

**Specimen Configuration**
- Wall dimension: 4 1/2 in. × 6 1/2 in.
- Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
- Fastener: #8-18x1” modified truss head self-drilling screw

Track: 50ksi, 362S150-54
Grain direction: Vertical

**Test results**
- Maximum load: 751.7 lbs
- Net lateral displacement at top of wall at Maximum load: 0.255 in.

**Observed Failure Mode:** Sheathing edge tear-out/bearing failure
Test Label: osb_54_track_vert #2

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Fastener: #8-18x1” modified truss head self-drilling screw

Track: 50ksi, 362S150-54
Grain direction: Vertical

Test results
Maximum load: 684.6 lbs
Net lateral displacement at top of wall at Maximum load: 0.398 in.

Observed Failure Mode: Sheathing bearing failure/semi pull-through
**Test Label: osb_54_track_vert #3**

**Specimen Configuration**
- Wall dimension: 4 1/2 in. × 6 1/2 in.
- Track: 50ksi, 362S150-54
- Grain direction: Vertical
- Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
- Fastener: #8-18x1” modified truss head self-drilling screw

**Test results**
- Maximum load: 753.8 lbs
- Net lateral displacement at top of wall at Maximum load: 0.385 in.

**Observed Failure Mode:** Sheathing bearing failure/semi pull-through and sheathing edge tear-out on the framing side
Test Label: osb_54_track_vert #4

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Fastener: #8-18x1” modified truss head self-drilling screw

Track: 50ksi, 362S150-54
Grain direction: Vertical

Test results
Maximum load: 700.7 lbs
Net lateral displacement at top of wall at Maximum load: 0.350 in.

Observed Failure Mode: Sheathing bearing failure/semi pull-through and sheathing tear-out on the framing side
Test Label: osb_54_track_vert #5

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Fastener: #8-18x1” modified truss head self-drilling screw

Track: 50ksi, 362S150-54
Grain direction: Vertical

Test results
Maximum load: 880 lbs
Net lateral displacement at top of wall at Maximum load: 0.251 in.

Observed Failure Mode: Shear failure in the screw
Test Label: osb_54_track_vert #6

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 24/16, Exposure I, 7/16 OSB
Fastener: #8-18x1” modified truss head self-drilling screw

Track: 50ksi, 362S150-54
Grain direction: Vertical

Test results
Maximum load: 756.5 lbs
Net lateral displacement at top of wall at Maximum load: 0.380 in.

Observed Failure Mode: Shear failure in the screw
Test Label: *ply_43_stud_horz #1*

**Specimen Configuration**
- Wall dimension: 4 1/2 in. × 6 1/2 in.
- Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
- Fastener: #8-18x1” modified truss head self-drilling screw
- Stud: 33ksi, 362S162-43
- Grain direction: Horizontal

**Test results**
- Maximum load: 884.8 lbs
- Net lateral displacement at top of wall at Maximum load: 0.544 in.

**Observed Failure Mode:** Sheathing bearing failure at the location of the screw/sheathing tear on the framing side
Test Label: ply_43_stud_horz_#2

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
Fastener: #8-18x1” modified truss head self-drilling screw

Stud: 33ksi, 362S162-43
Grain direction: Horizontal

Test results
Maximum load: 824.7 lbs
Net lateral displacement at top of wall at Maximum load: 0.552 in.

Observed Failure Mode: Sheathing bearing failure / semi pull-through
Test Label: ply_43_stud_horz_#3

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
Fastener: #8-18x1” modified truss head self-drilling screw

Specimen Configuration
Stud: 33ksi, 362S162-43
Grain direction: Horizontal

Test results
Maximum load: 869.8 lbs
Net lateral displacement at top of wall at Maximum load: 0.456 in.

Observed Failure Mode: Sheathing bearing failure/sheathing tear on the framing side
Test Label: ply_43_stud_horz_#4

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
Stud: 33ksi, 362S162-43
Fastener: #8-18x1” modified truss head self-drilling screw

Grain direction: Horizontal

Test results
Maximum load: 785.5 lbs
Net lateral displacement at top of wall at Maximum load: 0.448 in.

Observed Failure Mode: Sheathing bearing failure/tear along the grain through the location of the screw
Test Label: **ply_43_stud_horz_#5**

**Specimen Configuration**
- Wall dimension: 4 1/2 in. × 6 1/2 in.
- Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
- Fastener: #8-18x1” modified truss head self-drilling screw

**Test results**
- Maximum load: 686.2 lbs
- Net lateral displacement at top of wall at Maximum load: 0.392 in.

**Observed Failure Mode:** Sheathing bearing failure/tear along the grain through the location of the screw
Test Label: **ply_43_stud_horz_#6**

**Specimen Configuration**
- **Wall dimension:** 4 1/2 in. × 6 1/2 in.
- **Sheathing:** APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
- **Fastener:** #8-18x1” modified truss head self-drilling screw

**Test results**
- **Maximum load:** 739.9 lbs
- **Net lateral displacement at top of wall at Maximum load:** 0.531 in.

**Observed Failure Mode:** Sheathing bearing failure/semi pull-through
**Test Label: ply_43_stud_vert_#1**

**Specimen Configuration**
- Wall dimension: 4 1/2 in. × 6 1/2 in.
- Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
- Fastener: #8-18x1” modified truss head self-drilling screw
- Stud: 33ksi, 362S162-43
- Grain direction: Vertical

**Test results**
- Maximum load: 477.9 lbs
- Net lateral displacement at top of wall at Maximum load: 0.601 in.

**Observed Failure Mode:** Shear failure in the screw
Test Label: **ply_43_stud_vert_#2**

**Specimen Configuration**
- Wall dimension: 4 1/2 in. × 6 1/2 in.
- Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
- Fastener: #8-18x1” modified truss head self-drilling screw
- Stud: 33ksi, 362S162-43
- Grain direction: Vertical

**Test results**
- Maximum load: 728.6 lbs
- Net lateral displacement at top of wall at Maximum load: 0.343 in.

**Observed Failure Mode:** Sheathing tear through the location of the screw
Test Label: ply_43_stud_vert_#1

Specimen Configuration
Wall dimension: 4 1/2 in × 6 1/2 in.
Sheathing: APA rated, 32/16. Structural I, Exposure I, 15/32 4-Ply
Fastener: #8-18x1” modified truss head self-drilling screw

Stud: 33ksi, 362S162-43
Grain direction: Vertical

Test results
Maximum load: 803.2 lbs
Net lateral displacement at top of wall at Maximum load: 0.400 in.

Observed Failure Mode: Sheathing bearing failure/tear through the location of the screw
Test Label: **ply_43_stud_vert_#4**

**Specimen Configuration**
- Wall dimension: 4 1/2 in. × 6 1/2 in.
- Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
- Fastener: #8-18x1” modified truss head self-drilling screw

**Stud**: 33ksi, 362S162-43  
**Grain direction**: Vertical

**Test results**
- Maximum load: 480.5 lbs
- Net lateral displacement at top of wall at Maximum load: 0.220 in.

**Observed Failure Mode**: Shear failure in the screw
Test Label: ply_43_stud_vert_#5

Specimen Configuration
- Wall dimension: 4 1/2 in. × 6 1/2 in.
- Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
- Fastener: #8-18x1” modified truss head self-drilling screw

Stud: 33ksi, 362S162-43
Grain direction: Vertical

Test results
- Maximum load: 590.1 lbs
- Net lateral displacement at top of wall at Maximum load: 0.275 in.

Observed Failure Mode: Sheathing tear through the location of the screw
Test Label: ply_43_stud_vert_#6

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
Fastener: #8-18x1” modified truss head self-drilling screw

Stud: 33ksi, 362S162-43
Grain direction: Vertical

Test results
Maximum load: 535.8 lbs
Net lateral displacement at top of wall at Maximum load: 0.297 in.

Observed Failure Mode: Sheathing tear through the location of the screw
Test Label: `ply_43_track_horz_#1`

**Specimen Configuration**
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
Fastener: #8-18x1” modified truss head self-drilling screw

**Track:** 33ksi, 362T150-43  
**Grain direction:** Horizontal

**Test results**
Maximum load: 786.6 lbs  
Net lateral displacement at top of wall at Maximum load: 0.412 in.

**Observed Failure Mode:** Shear failure in screw
Test Label: ply_43_track_horz_#2

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
Fastener: #8-18x1” modified truss head self-drilling screw

Test results
Maximum load: 626.0 lbs
Net lateral displacement at top of wall at Maximum load: 0.373 in.

Observed Failure Mode: Shear failure in screw
Test Label: ply_43_track_horz_#3

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
Fastener: #8-18x1” modified truss head self-drilling screw

Test results
Maximum load: 748.5 lbs
Net lateral displacement at top of wall at Maximum load: 0.473 in.

Observed Failure Mode: Shear failure in screw
Test Label: ply_43_track_horz_#4

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
Fastener: #8-18x1” modified truss head self-drilling screw

Test results
Maximum load: 740.9 lbs
Net lateral displacement at top of wall at Maximum load: 0.431 in.

Observed Failure Mode: Shear failure in screw
**Test Label: ply_43_track_horz_#5**

**Specimen Configuration**
- Wall dimension: 4 1/2 in. × 6 1/2 in.
- Track: 33ksi, 362T150-43
- Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
- Grain direction: Horizontal
- Fastener: #8-18x1” modified truss head self-drilling screw

**Test results**
- Maximum load: 686.2 lbs
- Net lateral displacement at top of wall at Maximum load: 0.377 in.

**Observed Failure Mode:** Shear failure in screw
Test Label: **ply_43_track_horz_#6**

**Specimen Configuration**
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
Fastener: #8-18x1” modified truss head self-drilling screw

Track: 33ksi, 362T150-43
Grain direction: Horizontal

**Test results**
Maximum load: 725.4 lbs
Net lateral displacement at top of wall at Maximum load: 0.442 in.

**Observed Failure Mode:** Sheathing bearing failure / sheathing tears along the grain through the location of the screw
Test Label: ply_43_track_horz_#1

Specimen Configuration
Wall dimension: 4 1/2 in. × 6 1/2 in.
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
Fastener: #8-18x1” modified truss head self-drilling screw

Test results
Maximum load: 668.5 lbs
Net lateral displacement at top of wall at Maximum load: 0.408 in.

Observed Failure Mode: Sheathing bearing/tear-out failure
Test Label: ply_43_track_vert_#2

Specimen Configuration
Wall dimension: 4 1/2 in. x 6 1/2 in.
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
Fastener: #8-18x1” modified truss head self-drilling screw

Track: 33ksi, 362T150-43
Grain direction: Vertical

Test results
Maximum load: 616.4 lbs
Net lateral displacement at top of wall at Maximum load: 0.451 in.

Observed Failure Mode: Sheathing bearing/tear-out failure through the location of the screw
Test Label: ply_43_track_vert_#3

Specimen Configuration
Wall dimension: 4 1/2 in. x 6 1/2 in.
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
Fastener: #8-18x1” modified truss head self-drilling screw

Test results
Maximum load: 523.0 lbs
Net lateral displacement at top of wall at Maximum load: 0.318 in.

Observed Failure Mode: Shear failure in screw
Test Label: ply_43_track_vert_#4

Specimen Configuration
Wall dimension: 4 1/2 in. x 6 1/2 in.
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
Fastener: #8-18x1” modified truss head self-drilling screw

Test results
Maximum load: 722.7 lbs
Net lateral displacement at top of wall at Maximum load: 0.363 in.

Observed Failure Mode: Tilting/bearing failure, eventually a shear failure in screw
Test Label: **ply_43_track_vert_#5**

**Specimen Configuration**
Wall dimension: 4 1/2 in. x 6 1/2 in.
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
Fastener: #8-18x1” modified truss head self-drilling screw

**Track:** 33ksi, 362T150-43
**Grain direction:** Vertical

**Test results**
Maximum load: **561.1 lbs**
Net lateral displacement at top of wall at Maximum load: **0.286 in.**

**Observed Failure Mode:** Sheathing edge tear-out
Test Label: ply_43_track_vert_#6

Specimen Configuration
Wall dimension: 4 1/2 in. x 6 1/2 in.
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
Fastener: #8-18x1” modified truss head self-drilling screw
Track: 33ksi, 362T150-43
Grain direction: Vertical

Test results
Maximum load: 60.4.0 lbs
Net lateral displacement at top of wall at Maximum load: 0.317 in.

Observed Failure Mode: Shear failure in screw
APPENDIX D

DATA SHEETS OF PLYWOOD SHEATHED SHEAR WALL TESTS
Test Label: 4x8x43- PLY-2-M2

Test Date: August 16, 2011

Specimen Configuration
Wall dimensions: 4 ft. x 8 ft.  Studs: 33ksi, 362S162-43  Tracks: 33ksi, 362T150-43
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32-4-Ply
Fastener: #8-18x1” modified truss head self-drilling screw, 2in. o.c. on the perimeter, #10-16x1” on the hold-down.
Hold-down: S/HD10 welded on the left side
Environmental info: Humidity 38.5%, Temperature 25.3°C

Test protocol: Monotonic-ASTM E564

Test results
Maximum load: 10076 lbs
Net lateral displacement at top of wall at Maximum load: 3.39 in.

Observed Failure Mode: Plywood sheathing was raptured at the bottom and pulled off the frame at the center of the intermediate stud. Also, the intermediate stud buckled at the center of its length.
Test Label: 4x8x43- PLY-2-M2

Test Date: August 16, 2011

Specimen Configuration
Wall dimensions: 4 ft. x 8 ft.  Studs: 33ksi, 362S162-43  Tracks: 33ksi, 362T150-43
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
Fastener: #8-18x1” modified truss head self-drilling screw, 2in. o.c. on the perimeter. #10-16x1” on the hold-down.
Hold-down: S/HD10 welded  left side
Environmental info: Humidity 40.5%, Temperature 23.2°C

Test protocol: Monotonic-ASTM E564

Test results
Maximum load: 10456 lbs
Net lateral displacement at top of wall at Maximum load: 3.42 in.

Observed Failure Mode: Plywood sheathing pulled off the frame at the bottom region of the chord stud on the compression side. The chord stud on the compression side buckled at the bottom region.
Test Label: **4×8×43- PLY-6-M1**

**Test Date:** August 16, 2011

**Specimen Configuration**

Wall dimensions: 4 ft. × 8 ft.  
Studs: 33ksi, 362S162-43  
Tracks: 33ksi, 362T150-43  
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply  
Fastener: #8-18x1” modified truss head self-drilling screw, 6in. o.c. on the perimeter. #10-16x1” on the hold-down.  
Hold-down: S/HD10 / welded left side  
Environmental info: N.A.  

**Test protocol:** Monotonic-ASTM E564

**Test results**

Maximum load: 4616 lbs  
Net lateral displacement at top of wall at Maximum load: 2.09 in.

**Observed Failure Mode:** Plywood sheathing pulled off the frame along the bottom track and also along both of the chord studs at the lower half of the frame.
**Test Label: 4x8x43- PLY-6-M2**

**Test Date:** August 16, 2011

**Specimen Configuration**
Wall dimensions: 4 ft. x 8 ft.
Studs: 33ksi, 362S162-43
Tracks: 33ksi, 362T150-43
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
Fastener: #8-18x1” modified truss head self-drilling screw, 6in. o.c. on the perimeter. #10-16x1” on the hold-down.
Hold-down: S/HD10 / welded left side
Environmental info: N.A.

**Test protocol:** Monotonic-ASTM E564

**Test results**
Maximum load: 4273 lbs
Net lateral displacement at top of wall at Maximum load: 1.94 in.

**Observed Failure Mode:** One of the screws pulled off the top track at the top corner of the frame on the loading side. Screws tilted because of the steel framing bearing failure along the boundary studs.
Test Label: 4x8x43- PLY-2-C1

Test Date: August 17, 2011

Specimen Configuration
Wall dimensions: 4 ft. x 8 ft.  Studs: 33ksi, 362S162-43  Tracks: 33ksi, 362T150-43
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
Fastener: #8-18x1” modified truss head self-drilling screw, 2in. o.c. on the perimeter. #10-16x1” on the hold-down.
Hold-down: S/HD10
Environmental info: N.A.

Test protocol: Cyclic-CRUEE, reference displacement: 1.23 in. @ 43 cycles plus 3 cycles

Test results
Maximum +load: 10278 lbs
Net lateral displacement at top of wall at Maximum +load: 2.83 in.

Maximum -load: 10557 lbs
Net lateral displacement at top of wall at Maximum -load: 2.53 in.

Average maximum load: 10418 lbs
Average net displacement: 2.68 in.

Observed Failure Mode: Plywood sheathing fractured and the chord stud on the loading side was distorted at the top region. Bearing failure of sheathing at fasteners along the top track was evident and the sheathing edge was almost torn off.
Test Label: 4x8x43- PLY-2-C2

Test Date: August 19, 2011

Specimen Configuration
Wall dimensions: 4 ft. x 8 ft.  Studs: 33ksi, 362S162-43  Tracks: 33ksi, 362T150-43
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
Fastener: #8-18x1” modified truss head self-drilling screw, 2in. o.c. on the perimeter. #10-16x1” on the hold-down.
Hold-down: S/HD10
Environmental info: Humidity 43.4%, Temperature 22.9°C

Test protocol: Cyclic-CRUEE, reference displacement: 1.23 in. @ 46 cycles plus 3 cycles

Test results
Maximum +load: 11193 lbs
Net lateral displacement at top of wall at Maximum +load: 2.94 in.

Maximum -load: 10650 lbs
Net lateral displacement at top of wall at Maximum -load: 2.88 in.

Average maximum load: 10922 lbs
Average net displacement: 2.91 in.

Observed Failure Mode: Plywood sheathing fractured at the bottom corner on the loading side and finally pulled off the frame along the bottom track. Also, screws were pulled through the sheathing and some screw heads were sheared off along the bottom track.
Test Label: 4x8x43- PLY-4-C1-Retest

Test Date: October 21, 2011

Specimen Configuration

Wall dimensions: 4 ft. x 8 ft.  
Studs: 33ksi, 362S162-43  
Tracks: 33ksi, 362T150-43  
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply  
Fastener: #8-18x1” modified truss head self-drilling screw, 4in. o.c. on the perimeter. #10-16x1” on the hold-down.  
Hold-down: S/HD10  
Environmental info: Humidity 31.3%, Temperature 21.1°C

Test protocol: Cyclic-CRUEE, reference displacement: 2.8 in. @ 40 cycles

Test results

Maximum +load: 7165 lbs  
Net lateral displacement at top of wall at Maximum +load: 1.96 in.

Maximum -load: 6293 lbs  
Net lateral displacement at top of wall at Maximum -load: 1.86 in.

Average maximum load: 6729 lbs  
Average net displacement: 1.91 in.

Observed Failure Mode: Plywood sheathing pulled off the frame along the chord stud on the loading side, the top track on the loading side, and the bottom track because the screws were pulled though the sheathing and some screw heads were sheared off.
Test Label: 4x8x43- PLY-4-C2  Test Date: October 21, 2011

Specimen Configuration
Wall dimensions: 4 ft. x 8 ft.  Studs: 33ksi, 362S162-43  Tracks: 33ksi, 362T150-43
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
Fastener: #8-18x1” modified truss head self-drilling screw, 4in. o.c. on the perimeter. #10-16x1” on the hold-down.
Hold-down: S/HD10
Environmental info: Humidity 30.6%, Temperature 21.5°C

Test protocol: Cyclic-CRUEE, reference displacement: 2.8 in. @ 40 cycles

Test results
Maximum +load: 6868 lbs
Net lateral displacement at top of wall at Maximum +load: 1.94 in.

Maximum -load: 6908 lbs
Net lateral displacement at top of wall at Maximum -load: 1.87 in.

Average maximum load: 6888 lbs
Average net displacement: 1.91 in.

Oberved Failure Mode: Plywood sheathing pulled off the frame along the bottom track and chord studs at the lower half region because the screws were pulled though the sheathing and some screw heads were sheared off.
Test Label: 4x8x43- PLY-6-C1

Specimen Configuration
Wall dimensions: 4 ft. x 8 ft. Studs: 33ksi, 362S162-43 Tracks: 33ksi, 362T150-43
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
Fastener: #8-18x1” modified truss head self-drilling screw, 6in. o.c. on the perimeter. #10-16x1” on the hold-down.
Hold-down: S/HD10
Environmental info: Humidity 38.9%, Temperature 22.6°C

Test protocol: Cyclic-CRUEE, reference displacement: 1.23 in. @ 49 cycles

Test results
Maximum +load: 4533 lbs
Net lateral displacement at top of wall at Maximum +load: 1.82 in.

Maximum -load: 3694 lbs
Net lateral displacement at top of wall at Maximum -load: 1.51 in.

Average maximum load: 4114 lbs
Average net displacement: 1.67 in.

Observed Failure Mode: Plywood sheathing pulled off the frame along the chord stud on the loading side because the screws were pulled though the sheathing and some screw heads were sheared off. One of the screws that were connecting the loading T-beam to the top track was pulled off.
Test Label: 4x8x43- PLY-6-C1  Test Date: September, 2011

Specimen Configuration
Wall dimensions: 4 ft. x 8 ft  Studs: 33ksi, 362S162-43  Tracks: 33ksi, 362T150-43
Sheathing: APA rated, 32/16, Structural I, Exposure I, 15/32 4-Ply
Fastener: #8-18x1” modified truss head self-drilling screw, 6in. o.c. on the perimeter. #10-16x1” on the hold-down.
Hold-down: S/HD10
Environmental info: Humidity 40.5%, Temperature 23.2°C

Test protocol: Cyclic-CRUEE, reference displacement: 1.23 in. @ 49 cycles

Test results
Maximum +load: 4542 lbs
Net lateral displacement at top of wall at Maximum +load: 1.73 in.

Maximum -load: 3395 lbs
Net lateral displacement at top of wall at Maximum -load: 1.83 in.

Average maximum load: 3969 lbs
Average net displacement: 1.78 in.

Observed Failure Mode: Plywood sheathing fractured in the middle at the lower half region and finally pulled off the frame along the bottom track, intermediate stud, and the chord stud on the loading side because the screws were pulled though the sheathing and some screw heads were sheared off. Also, some screws pulled off the T-beam connected to the top track.
REFERENCES

ACI 318 (2005), “Building Code Requirements for Structural Concrete and Commentary”, American Concrete Institute, Detroit, MI.


168


Yu, C. (2007). ”Steel Sheet Sheathing Options for Cold-Formed Steel Framed Shear Wall Assemblies,” Research Report RP07-3 submitted to American Iron and Steel Institute, Washington, DC.